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Geotechnical engineering applications in opencast coal mining – case studies from Northern England

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of Doctor of Philosophy

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APPENDICES B and C

APPENDIX B

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PAPER B1 TILLS FRAMEWORK

Clarke, B.G., Aflaki, E. and Hughes, D.B. 1997.

A framework for characterisation of glacial tills.

Proceedings of 14th International Conference on Soil Mechanics and Foundation Engineering, Hamburg, 263-266.

A framework for characterization of glacial tills

Un cadre pour caractériser les tills glaciaires

B.G.Clarke, E.Aflaki & D.Hughes – *University of Newcastle upon Tyne, UK*

ABSTRACT: Tills are extremely variable soils which were gravitationally compacted and possibly sheared when deposited. They are overconsolidated but not necessarily due to ice because any pore pressure regime would have been affected by the temperature profile through the till and the underlying rock characteristics. The in situ properties together with intrinsic properties can be used to develop a geotechnical model to provide a consistent framework to produce design parameters. One dimensional compression and swelling tests and effective stress triaxial tests on normally and over- consolidated tills have been used to determine the intrinsic properties of compression, swelling and strength. The normal compression data lie on the intrinsic compression line. The swelling data, when plotted in terms of overconsolidation ratio and intrinsic swelling index, lie within a narrow band. These intrinsic compression and swelling characteristics have been used to develop a relationship between undrained strength and activity, and to normalise the failure envelopes.

RESUME: Les tills, ayant été compactés par la gravitation et peut-être fracturés lors de leur dépôt, sont des terrains extrêmenent hétérogènes. Ces terrains sont sur-consolidés mais pas nécessairement à cause de la glace puisque le régime de pression interstitielle change en fonction du profil de température au travers des tills et des caractéristiques des roches sous-jacentes. Les propriétés in situ et les propriétés intrinsèques peuvent être utilisées pour développer un modèle géotechnique fournissant un cadre fiable qui offre des paramètres de définition. Des tests unidimensionnels de compression et de gonflant, et des tests triaxiaux de contraintes réelles sur des tills normaux et des tills sur-consolidés ont été utilisés pour déteminer les propriétés intrinsèques de compression, expansion et solidité. Les données de compression normale se trouvert sur la ligne de compression intrinsèque. Les données d'expansion, tracées point par point en fonction de la proportion de sur-consolidation et de l'index d'expansion intrinsèque, se situent dans une bande étroite. Ces caractéristiques intrinsèques de compression et d'expansion ont été utilisées pour devélopper un rapport entre la résistance au cisaillement sans assèchement et l'activité, et pour normaliser les enveloppes de rupture.

1. INTRODUCTION

Over 60% of the British Isles was covered by an ice sheet at sometime resulting in extensive glacial deposits which are some of the most widely distributed deposits in the world. The most common of these deposits, glacioterrestrial deposits, were formed either subglacially as lodgement till or deformation till or, deposited from within the ice of a retreating glacier in the form of melt out till or flow till. The lodgement tills are typically heterogeneous mixtures of clays, silts, sands, gravels and cobbles. Melt out tills are subglacial, supraglacial or englacial deposits which can be similar to lodgement till but can also include extensive lenses of clays, sands and gravels. Thus, melt out tills and lodgement tills may have a similar composition but their engineering properties could differ because of the different methods of deposition.

Thus, design parameters should be selected from a study of the geotechnical processes and as well as taken from results of mechanical tests. Quality samples for design parameters are usually restricted to the clay matrix dominant till which is free of cobbles. Any gravel present, which is usual, may affect the sampling, and deformation and failure of a specimen. This can lead to a range of values for a given property making it difficult to select design parameters. For example, Figure 1 shows a profile of undrained shear strength of a lodgement till identified at one site.

Soil models can be used as a framework to take into account the natural variability but conventional models cannot explain the behaviour of till. For example, it is difficult to explain shear strengths equivalent to those of weak rock within a few metres of the ground surface, a common feature of lodgement tills.

Thus, tills are non-textbook materials that should be treated as difficult ground. There is a need for a geotechnical model to describe the properties of and classify till, allowing a consistent framework to be used in the interpretation of site investigation data. This model should be based on an understanding of the depositional and post depositional processes as well as mechanical tests to determine the stress-strain-time relationships.

2. FORMATION OF TILL

Till was formed either by gravitational compaction and shearing (lodgement and deformation tills) which took place as the ice

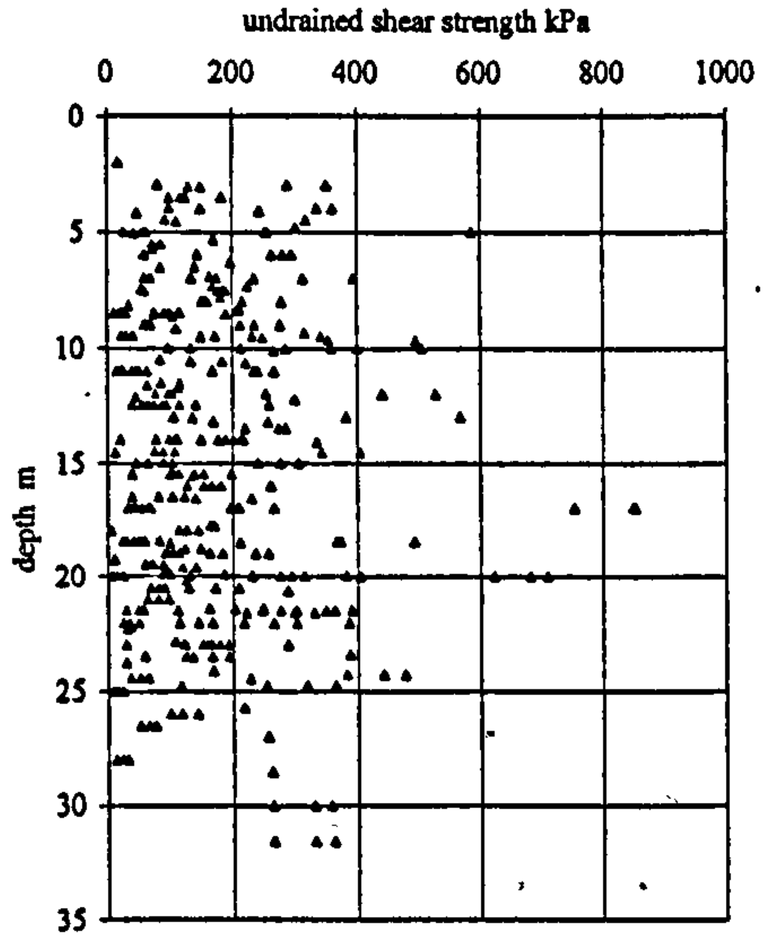

Figure 1 A profile of undrained shear strength of a lodgement till

advanced, or by gravitational compaction (melt out and flow tills) which took place as the ice or retreated. At any particular location several different processes could have taken place due to successive periods of glaciation and interstadial flow. Further, a number of pore pressure regimes, Table 1, could have existed depending on whether the glacier was formed of temperate or cold ice (Boulton et al, 1977), the permeability of the underlying rock, and the temperature profile through the till. As the ice retreated (melted) any excess pore pressures would have dissipated due to relief of load. An implication is that till may not be heavily overconsolidated as suggested by the weight of ice.

Table 1 Pore pressure regimes within till during deposition

Glacier	Till	Underlying Rock	Pore Pressure
frozen	base unfrozen	aquifer	no excess
frozen	base unfrozen	no aquifer	excess
base unfrozen	unfrozen	aquifer	no excess
base unfrozen	unfrozen	no aquifer	excess
frozen	frozen	not relevant	none

Glacial velocities exceeding 50-100 m/year would be sufficient to erode an existing till (Boulton et al, 1977). Thus, it is likely that tills have been reworked and redeposited a number of times during successive periods of glaciation such that most of the deposits identified today were laid down during the last glacial period.

Thus, till is a complex deposit which contains material from sources remote from the place of deposition. Lodgement till is the till that has been subjected to the greatest amount of compaction, shearing and transportation. Deformation till has also undergone gravitational compaction and shear but it is primarily formed of the underlying deposits which could include till and rock. Thus deformation till, as far as engineering properties are concerned, may be considered a lodgement till which has undergone limited movement and hence shear. It can, however, contain traces of bedrock in the form of slabs which could be mistakenly identified as rockhead (Hughes et al, 1996).

Melt out till is usually deposited during the retreat of a glacier and is therefore only subjected to gravitational compaction. Elements of that till, when being transported within a glacier, may be subjected to gravitational compaction and shearing. Melt out tills are likely to be more variable than lodgement tills since they can contain glaciolacustrine and fluvioglacial deposits, that is intraformational sands, gravels and laminated clays which have been deposited in subglacial or englacial meltwater channels and lakes.

Tills will have undergone ageing and weathering which may have changed the properties created during deposition. The effects of weathering would decrease with depth and be influenced by any extensive lenses of more permeable material which might have acted as barriers to the process of weathering.

In conclusion, till may have been gravitationally compacted, sheared, possibly reworked and weathered. Till has not been deposited under water therefore is not a sedimented deposit. The implication is that the concept of consolidation of soils on which much of our knowledge of soil mechanics theory is based may not be valid.

3. TILLS OF NORTH EAST ENGLAND

In order to develop an understanding of the effects depositional and post depositional processes have upon till, a pilot study of the tills of North East England has been instigated. It is generally accepted in the UK there is often a tripartite succession of till and in this area it is identified as an upper till separated from a lower till by discontinuous layers of gravels, sands and lacustrine clays. The upper till is divided into an uppermost mottled till and a red till. The tills have been considered as either two or more separate lodgement tills (Beaumont, 1968), a lodgement till overlain by a

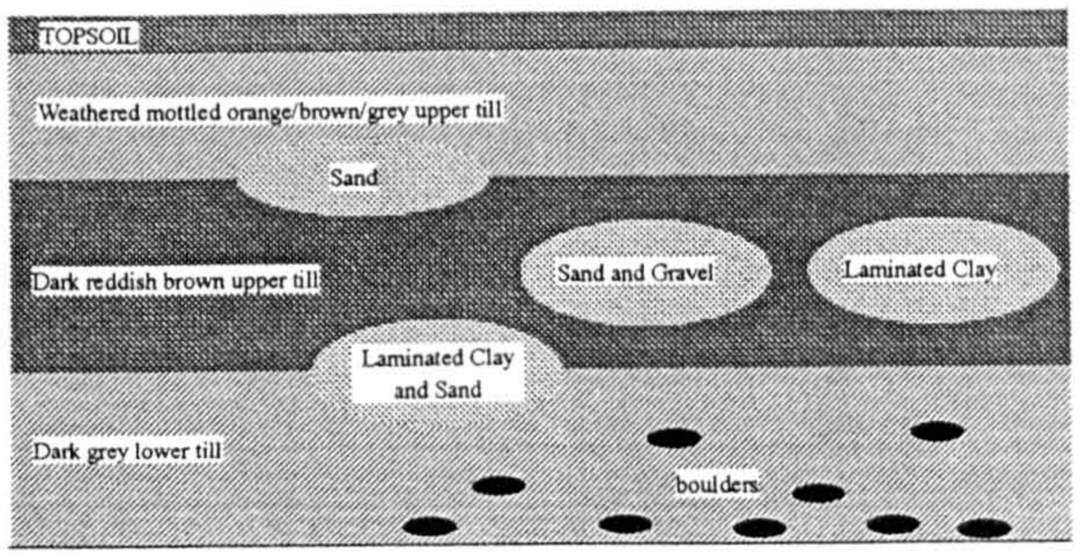

Figure 2 A geological model for the tills of North East England (after Robertson et al, 1994)

melt out till (Carruthers, 1953) or a single lodgement till with a weathering profile down to the discontinuity between the two tills (Eyles and Sladen, 1981).

Studies of borehole records and exposures in opencast mines have been used to develop the geological model given in Figure 2. It shows an upper clay till containing lenses of glaciolacustrine and fluvioglacial deposits overlying the lower stiffer grey till. Generally the strength, stiffness and granular content increase with depth; the plasticity and permeability decrease with depth.

These tills are typically stiff to very stiff sandy clay with gravel, containing some boulders and lenses of sand and gravel and laminated clay. Laboratory tests are mostly carried out on the clay matrix because of the difficulty of sampling the more granular component of the till. An implication of this is that the results obtained represent the strength and stiffness of the "softer" component of the till which may be one reason for the difference between laboratory and in situ strengths. Generally the in situ strength of till exceeds that measured in a laboratory which can give rise to contractual problems. The range of geotechnical properties shown in Table 2 indicates the variability arising because of the heterogeneous nature of these tills and the effects the particles have upon the samples and their properties.

4. INTRINSIC PROPERTIES OF TILLS

Burland (1990) suggested that a study of the intrinsic properties of a reconstituted soil would provide a robust framework for the interpretation of site investigation data. This may be particularly important when dealing with till, a difficult non-textbook soil as

Table 2 Classification properties of tills of North East England

Property		Upper Till		Lower Till	
		sample	in situ	sample	in situ
Natural w	%	15.9	9-31	20.6	9-23
c_u	kPa		50-410[2]		65-410[2]
LL	%	57.0		31.1	
PL	%	25.6		17.1	
PI	%	31.4		14.0	
Clay Fraction	%	38.6		22.6	
Silt Fraction	%	44.5		34.2	
Sand Fraction	%	13.5		31.6	
Gravel Fraction	%	2.1		8.5	
Kaolinite	%		54-63[1]		67-70[1]
Illite	%		14-29[1]		22-36[1]
Quartz	%		45-52[1]		53[1]
Activity	%	0.81	0.65-0.70	0.62	0.59-0.67
Specific Gravity		2.69		2.65	
Density	Mg/m^3		1.62-1.93[2]		1.76-2.00[2]
c'	kPa	4.7	0-25[3]	2.3	0-15[3]
ϕ'		22.5	27-35[3]	26.3	32-37[3]

Source:
1. Thabet (1973), 2. Robertson et al (1994), 3. Eyles and Sladen (1981)

described above. In this study, disturbed samples of upper and lower till were obtained from a major opencast site. Particles greater than 2 mm in diameter were removed to produce reconstituted tills which had the properties given in Table 2. A fabric free slurry was made by increasing the water content of the natural till until it was 150% of the liquid limit. The slurry was allowed to consolidate under a predetermined pressure. While this does not represent the processes involved in the formation of till, that is gravitational compaction and shearing, it does produce a consistent material so that the effects of particle size and range, void ratio and particle type on strength and stiffness can be found.

4.1. *Compression Characteristics*

One dimensional compression and swelling characteristics were determined over a pressure range of 6 kPa to 3200 kPa. Three dimensional compression and swelling characteristics were found from triaxial tests in which isotropic effective stresses of up to 5 MPa were applied. Sub-samples of compressed slurry with overconsolidation ratios (OCR) varying between 1 and 10 were used for triaxial shearing. These encompassed the range of OCRs determined from tests on intact specimens which do not conform with the suggested ice thickness of 1.4 km (Boulton et al, 1977) giving some support to the concepts given in Table 1.

Burland (1990) suggested that a unique intrinsic compression line, ICL, given by Equation (1) exists for all clays. This line is defined in terms of the intrinsic void index, I_v, and the vertical effective stress used to compress the specimen one dimensionally.

$$I_v = 2.45 - 1.285x + 0.015x^3 \qquad (1)$$

where x is $\log_{10}(\sigma'_v)$ and I_v is defined in Equation (2) in terms of the current void ratio, e, and the void ratios at 100 kPa, e^*_{100}, and 1000 kPa, e^*_{1000}.

$$I_v = \frac{e - e^*_{100}}{e^*_{100} - e^*_{1000}} \qquad (2)$$

The one dimensional compression curves for the tests on the tills conform with the ICL, between 50 and 3200 kPa (Clarke et al, 1996).

Clarke et al (1996) have shown that data from the isotropic compression tests also conform with Equation (1) but only over a stress range of 100 kPa to 1000 kPa (Figure 3). Over the full range of applied stress, 100 kPa to 5400 kPa, a straight line is a better fit such that:

$$I_v = 2 - \log_{10}\sigma'_v \qquad (3)$$

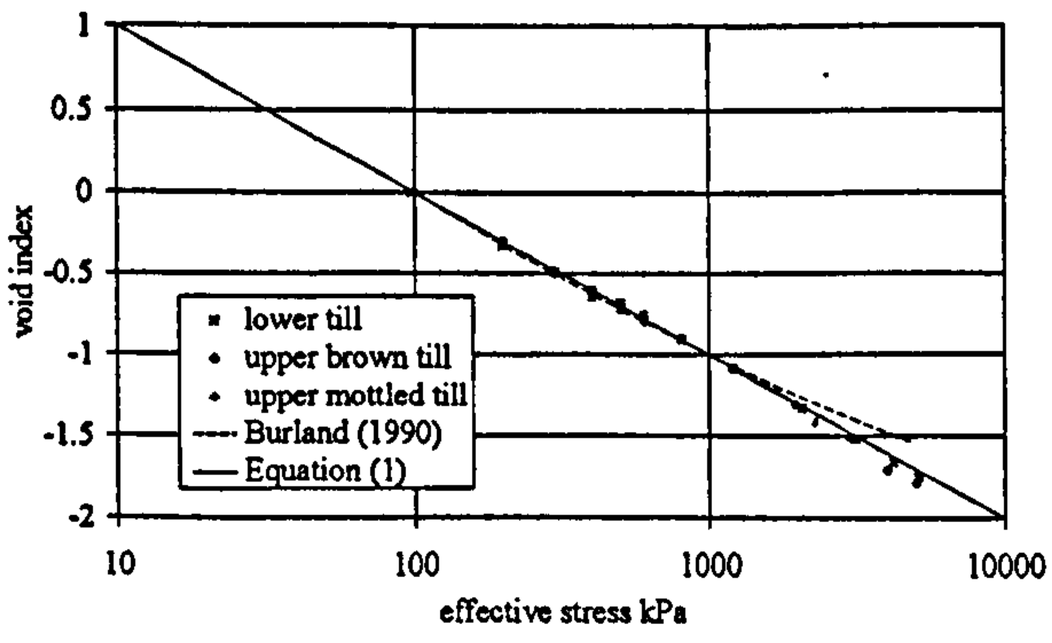

Figure 3 The intrinsic compression line for isotropic compression (after Clarke et al, 1996)

Clarke et al (1996) have suggested a similar definition can be used for an intrinsic swelling void index, I_{vs}, which is expressed in terms of the void ratios at effective vertical pressures at OCRs of 2, $e^*_{pc/2}$, and 20, $e^*_{pc/20}$, such that:

$$I_{vs} = \frac{e - e^*_{pc/2}}{e^*_{pc/20} - e^*_{pc/2}} \qquad (4)$$

A plot of I_{vs} against OCR, Figure 4, shows that the data from the one dimensional and isotropic swelling tests lie within a band. This band is centred on a line, Equation (5), of a similar form to the ICL but expressed in terms of x which is equal to $\log_{10}$(OCR).

$$I_{vs} = -0.242 + 0.738x + 0.196x^2 - 0.028x^3 \qquad (5)$$

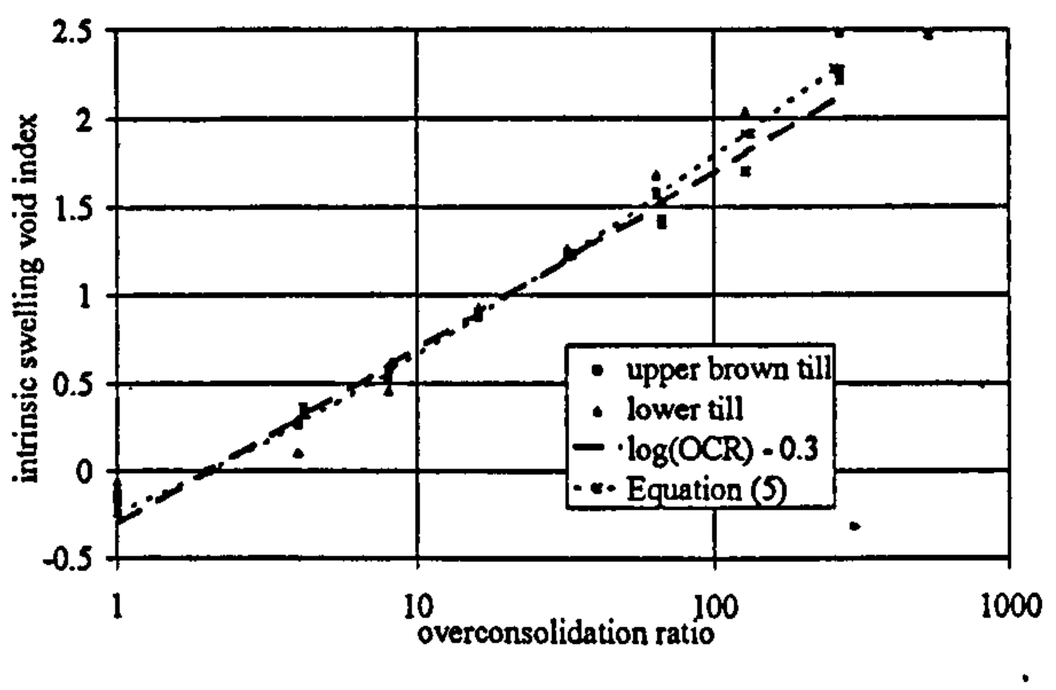

Figure 4a The one dimensional intrinsic swelling band (after Clarke et al, 1996)

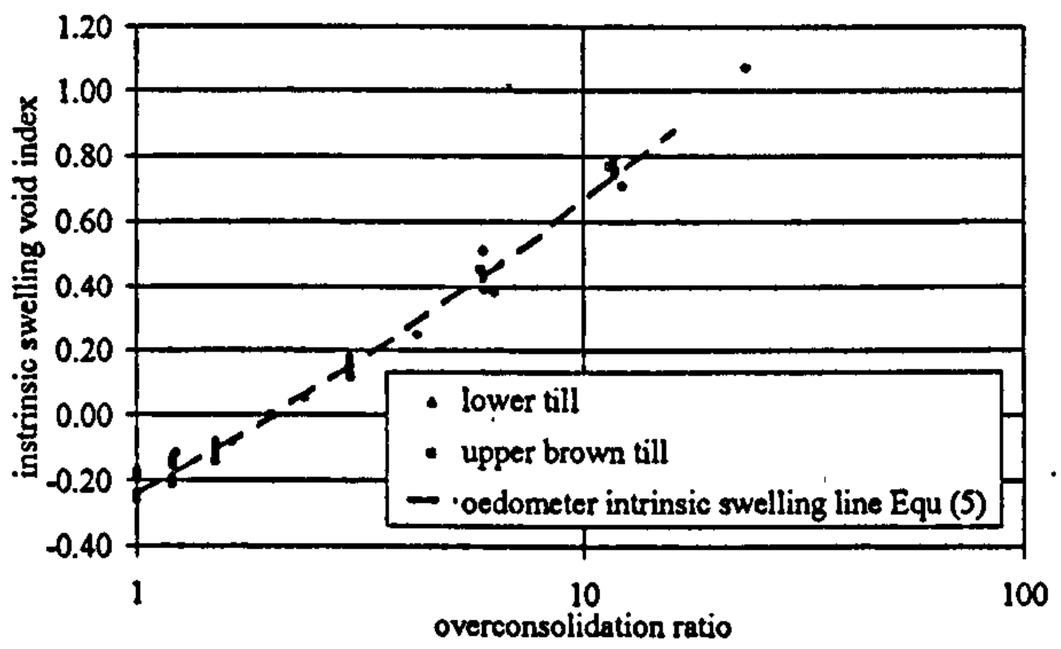

Figure 4b The isotropic intrinsic swelling band (after Clarke et al, 1996)

4.2 *Strength Characteristics*

A series of consolidated undrained triaxial tests were carried out on reconstituted upper and lower till. Burland et al (1997) suggested that an intrinsic failure envelope could be defined as:

$$\frac{t}{\sigma_{ve}^*} = \kappa^* + \frac{s'}{\sigma_{ve}^*}\tan\phi_e' \qquad (6)$$

where σ_{ve}^* is the effective vertical pressure on the intrinsic compression line at the same void ratio as the test specimen, ϕ_e^* is the intrinsic angle of shearing resistance, κ^* the intrinsic cohesion and, t and s' are the two dimensional stress invariants.

Figure 5 shows the peak shear stress, t, from the tests on the overconsolidated reconstituted till plotted in accordance with Equation (5). The lines represent the intrinsic Hvorslev surfaces

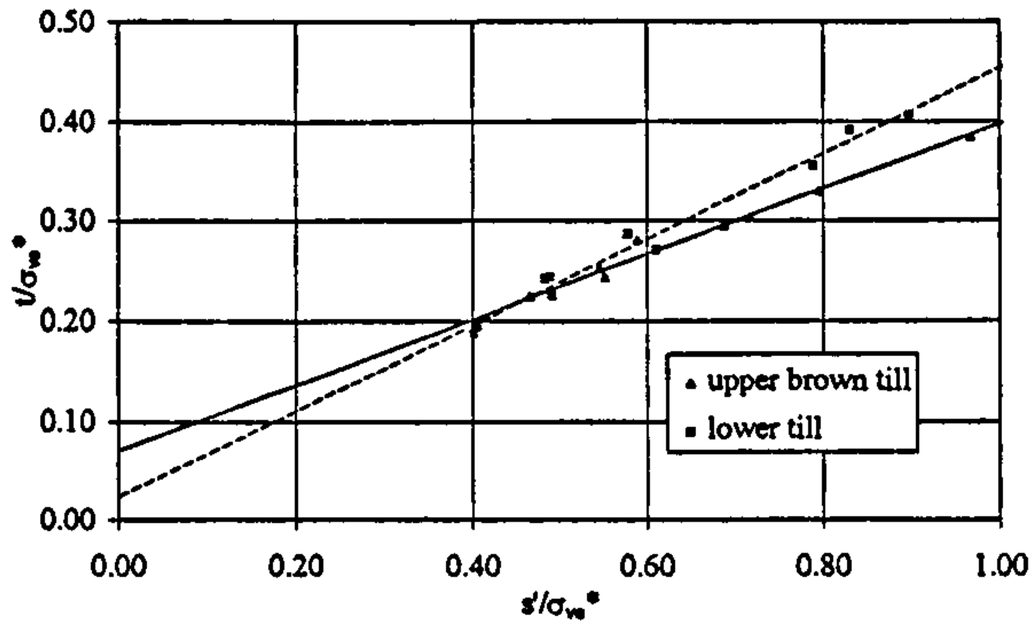

Figure 5 The intrinsic Hvorslev failure surface for reconstituted upper and lower tills

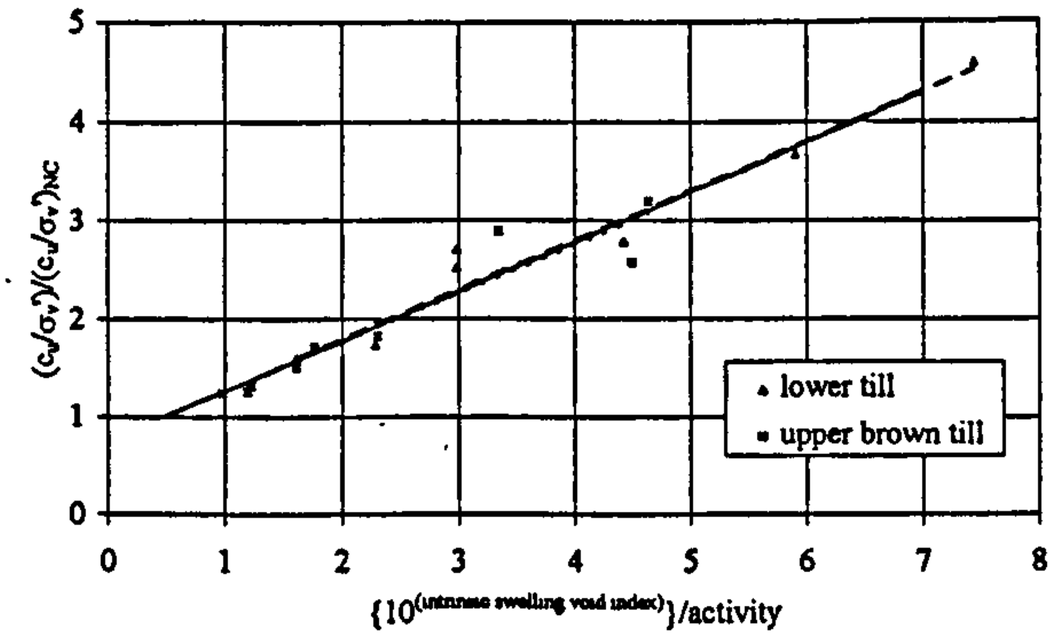

Figure 6 The variation in undrained strength with clay type, overconsolidation ratio and effective stress

for the two tills.

The upper limit to Equation (5) is the intrinsic critical state line, CSL, which can be defined in terms of (t/σ^*_{ve}) and (s'/σ^*_{ve}). The intrinsic angle of shearing resistance for the CSL, ϕ^*_{cs}, was found from tests on normally consolidated reconstituted tills.

Much use is made of the undrained shear strength in design, therefore a consistent method to select a design profile from data such as those shown in Figure 1 would be useful. The scatter in the data may be attributed to the variation in the clay content, fabric, gravel content, type of clay, water content, degree of weathering and depositional processes. The results of tests on reconstituted till should only depend on the type of clay, void ratio and stress history.

Figure 6 shows the peak undrained shear strength from tests on overconsolidated reconstituted tills normalised with respect to the consolidation pressure and the mean value of that ratio for the normally consolidated till, $(c_u/\sigma'_v)_{NC}$, plotted against a function of the activity of the till and the intrinsic swelling void index derived from Equation (5). The data for both tills lie about straight lines which are very nearly concurrent suggesting that a unique line, Equation (7) may be developed.

$$\frac{c_u / \sigma'_v}{[c_u / \sigma'_v]_{NC}} = 0.504 \frac{10^{I_{vs}}}{\text{activity}} + 0.77 \qquad (7)$$

The intrinsic properties of the upper and lower tills are given in Table 3. These represent properties of the tills which are independent of fabric and, in the case of strength where the data are normalised with respect to σ^*_{ve}, the void ratio. In order to make use of these data it is necessary to study in detail the properties of intact till. In practice, however, application of these properties in geotechnical design, such as stability analyses, should produce a safe solution since they represent a lower bound.

Table 3 Intrinsic properties of tills of the North East of England

Property	Upper Brown Till	Lower Till
e^*_{100} (NC)	1.044	0.661
e^*_{1000} (NC)	0.720	0.460
C_c^* (NC)	0.324	0.201
ϕ^*_{cs}	21	24
C_s^*	0.076	0.055
ϕ'^*_p	18	23
κ^*	0.071	0.025

5. CONCLUSIONS

Glacioterrestrial deposits have been gravitationally compacted and sheared during deposition, and then aged and possibly weathered. They are insensitive and very stiff but may not be heavily overconsolidated. A description of a till based on structure and fabric should be extended to include the type of till, as tills of different origin may have the same classification.

One dimensional and isotropic tests on reconstituted tills show that the compression characteristics conform with the ICL. Further, the swelling data for overconsolidated specimens lie within an intrinsic swelling band which is expressed in terms of an intrinsic swelling index and overconsolidation ratio.

Consolidated undrained triaxial tests on specimens of normally and over- consolidated reconstituted tills enabled an intrinsic Hvorslev failure line to be developed and a relationship between undrained shear strength, intrinsic swelling index and activity to be established.

A description of a till including its origin and intrinsic parameters of compression, swelling and strength are to be used as a geotechnical model to provide a consistent framework for the interpretation of site investigation data from tests on this extremely variable non-text book material.

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PAPER B2 GLACIAL HISTORY

Teasdale, D.A. and Hughes, D.B. 1999.

The glacial history of north-east England.

The Quaternary of North-east England Field Guide (eds. D.R. Bridgland, B.P. Horton and J.B. Innes), Quaternary Research Association, 10-17.

The Glacial History of north-east England.

D. Teasdale & D. Hughes

North-east England is bounded on the north by the Cheviot Hills, the summit of the Pennine escarpment to the west, the North Yorkshire Moors to the south and the present North Sea coastline to the east. The area is favoured with excellent, easily accessible Quaternary sections along its entire coast. Valley exposures also occur, especially where Post-glacial isostatic uplift has caused valley incision, for example along much of the Durham coastline. Additional data is obtainable from the extensive records collected during deep and opencast coal-mining activities, from geotechnical boreholes associated with civil engineering projects, and from offshore boreholes and seismic studies.

North-east England is a mainly low-lying area, which at the present day experiences a rain-shadow effect caused by the higher ground to the north and west. During the Late Devensian, ice never accumulated in significant amounts within the region, although small ice caps may have existed on the highest areas of the Pennines (Lunn, 1995) and Cheviot Hills (Clapperton, 1970). Evidence from the extensive glacial deposits of the region indicates that ice did pass over north-east England on its way elsewhere. Clast lithological studies indicate that the over-riding ice came from a number of sources, including the Scottish Highlands, the Lake District and the Southern Uplands of Scotland. Evidence from older pre-Late Devensian deposits found in a narrow strip along the Durham coast indicate an easterly ice source, possibly from as far away as Scandanavia. Later, during the Late Devensian, the entire region was covered by ice originating to the north and west of the region. Finally, there is evidence of northern ice moving southwards parallel to the coast and sometimes onshore. This late event may represent a North Sea ice lobe surging into the southern North Sea. Early interpretations which relied on evidence for multiple periods of ice-sheet growth and decay have subsequently been cast into doubt by the attribution of almost all of the deposits seen to Late Devensian events.

Despite over one hundred and thirty years of continuing research into the glacial history of the north-east England, no single model has been advanced which explains the available evidence satisfactorily. This contribution will outline four of the key issues yet to be resolved. For more comprehensive discussions and reviews of previous research, the reader is referred to Beaumont (1968), Douglas (1991), Lunn (1995), Hughes *et al.* (1998) and Mills and Holliday (1998).

Pre-Late Devensian Events

The main line of evidence that points to differing styles of glaciation in north-east England before the Late Devensian is the presence of unusual clast lithologies. These indicate that a narrow coastal strip had been covered by ice originating from much further east. Boulders of larvikite, originating in Norway, were reported by Trechmann (1915) along the Durham coastline. The best example was at Warren House Gill (NZ 448423), in which more than 80% of the clasts were igneous or metamorphic rocks from the Christiana district

of southern Norway. Beaumont (1967) reported that Trechmnann had found some 6% of the erratics in the deposit to be 'chalk and flint, and red and green Triassic material, which could have been derived from the floor of the North Sea'. More recently, Beaumont (1967) was able to examine a poor exposure about 1m high, along the northern bank of Warren House Gill, and came to slightly different conclusions to Trechman. The Scandinavian Drift was found to contain very little material which had been derived from the floor of the North Sea, or from the Durham coastal area. Carboniferous Limestone, coal and ironstone were present in overlying deposits at higher levels in the section. Beaumont concluded that the Scandinavian Drift must predate the arrival of later ice which brought Carboniferous Limestone and other rocks from the north and west.

Beaumont (1967, p.441) noted that "the problem is to decide whether this clay is the subglacial till of a completely terrestrial ice sheet which has advanced across the floor of the North Sea, or whether the deposit was produced by the melting of frozen debris in the sole of a cold based ice shelf which had floated across the North Sea and grounded along the Durham coast". Whilst Trechmann envisioned grounded ice across the entire North Sea Basin, Beaumont preferred the latter interpretation. Sejrup *et al.* (1994) have suggested that the Central North Sea has remained ice free since about 26 Ka BP so a grounded ice origin for this deposit would mean it was of at least that age. An ice shelf origin could equally be of Late Devensian age or older.

The age of the Scandinavian Drift is of some importance, because it has been correlated by Trechmann (1915) and Catt (1991a) with the Basement Till of Yorkshire. This correlation was made on the basis of shells distributed throughout the more clayey portions of the Scandanavian Drift, which bear "a rather close resemblance to that recorded from transported shell beds in the Basement Clay at Flamborough South Landing" (Lamplugh, 1889a). The age of the Basement Till in Yorkshire is, however, the subject of recent debate. Eyles *et al.* (1994) proposed that the Basement Till was Devensian in age, whilst Bateman and Catt (1996) suggested a pre-Ipswichian age, based on an OIS 5e age inferred for the overlying Sewerby Raised Beach. Having reliable dates for the Scandinavian Drift would shed light upon the very poorly understood Early and Middle Devensian periods across the whole North of England. See also discussion of the age of the Easington Raised Beach, which incorporates glacially-derived rocks of both British and Scandinavian origin (Bridgland, this volume: Bridgland and Austin, this volume).

North-east England has probably experienced total inundation by ice at least twice during the Quaternary, but the sedimentary evidence that remains from early events is limited. Early Quaternary sediments were probably highly eroded and re-worked by the Late Devensian glaciation. Although the possibility exists that pre-Late Devensian sediments may be extensive, only fragmentary evidence for any interglacial sediments or weathering profiles exists, and so no firm conclusions can be drawn. Direct dating of any cold-stage sediments has not been undertaken in north-east England. This absence of reliably dated material means that the length or importance of any unconformities identified in Quaternary sections within the region is unknown. Indirect evidence is all that is available to shed light on the presumed ages of pre-Late Devensian sediments, including the Scandinavian Drift.

The North Sea Lobe of the British Ice Sheet

During and after the Last Glacial Maximum, ice that had accumulated in the Lake District and Southern Uplands entered north-east England from the west through breaches in the Pennine escarpment, especially at the Stainmore Pass and the Tyne Gap. The Late Devensian ice-limit occurs to the south of this region, in the Vale of York. The ice was flowing generally east or south-eastwards at the Last Glacial Maximum, but as the ice eventually thinned it began to follow the influence of the underlying relief. The preserved glacial sediments often record the last events in the glaciation and indicate the strong influence of local topography. As the western ice waned, it became confined to progressively lower parts of the valleys where extensive valley train sands and gravels were deposited, as for example along the Tyne valley between Ryton (NZ 170627) and Wylam (NZ 128638).

It has been proposed by various workers that sediments found along the coast of north-east England record one or more, probably late, North Sea lobes of the British ice sheet flowing southwards parallel to the coast. Boulton *et al.* (1977) were unable to model the British Ice Sheet successfully without envisioning a surging ice lobe in the North Sea region during the Late Devensian. Lambeck's (1993a, 1993b) models went further in predicting that the ice-flow or surge was a relatively short-lived affair, which occurred between 22 Ka BP and shortly after 18 Ka BP. Eyles *et al.* (1994) speculated that the till sheets of East Yorkshire have an offlapping geometry towards the east due to a recurrently surging ice lobe reaching less far inland during succesive surges. Figure 8 shows a composite model of the maximum extent of this North Sea lobe, based particularly on Balson and Jeffery (1981). Ice from the North Sea Lobe can be traced from the Firth of Forth and Cheviot Hills past the coast of north-east England and along the Yorkshire coast, before terminating to the northeast of Norfolk.

The relative timing of the growth of the North Sea ice lobe in relation to that of ice sourced to the west of the region has been the subject of much speculation. Raistrick's (1931) model of ice flow patterns across north-east England showed ice flowing in streams from accumulation areas that varied in importance through time. During early periods of the last glaciation, western ice dominated, whereas at later stages southward flowing Cheviot or Tweed Ice was dominant. Lunn (1995) notes the southward deflection of Cheviot and Tweed ice along the coastal belt, presumably by a North Sea ice lobe (Figure 8 also shows the directions of ice flow). The coastal ice only came so far inland, but the question remains, how far?

It has been suggested that the western ice withdrew from much of north-east England before the North Sea lobe, thus creating an ice-free zone between glaciers occupying valleys and a lobe close to the present coastline. Whilst the western ice was producing large quantities of meltwater, the coastal ice thick enough to present an impermeable barrier to water flowing to the east. Many workers, including Smythe (1902) and Smith (1981, 1994) have envisioned the coastal ice forming ephemeral preglacial lakes against higher ground to the west, which in turn led to the deposition of extensive lacustrine sedimentary bodies. Published Drift Geology maps from the region show extensive sand and gravel bodies occupying many valleys between 40-100m OD. Hunting for lakes and correlating between

them has been very popular at times. Smith (1994) believed that almost all of the lowlands around Sunderland were at one time covered by such a lake, Glacial Lake Wear, which deposited the widespread Tyne-Wear Complex deposits (see Hughes and Teasdale, this guide).

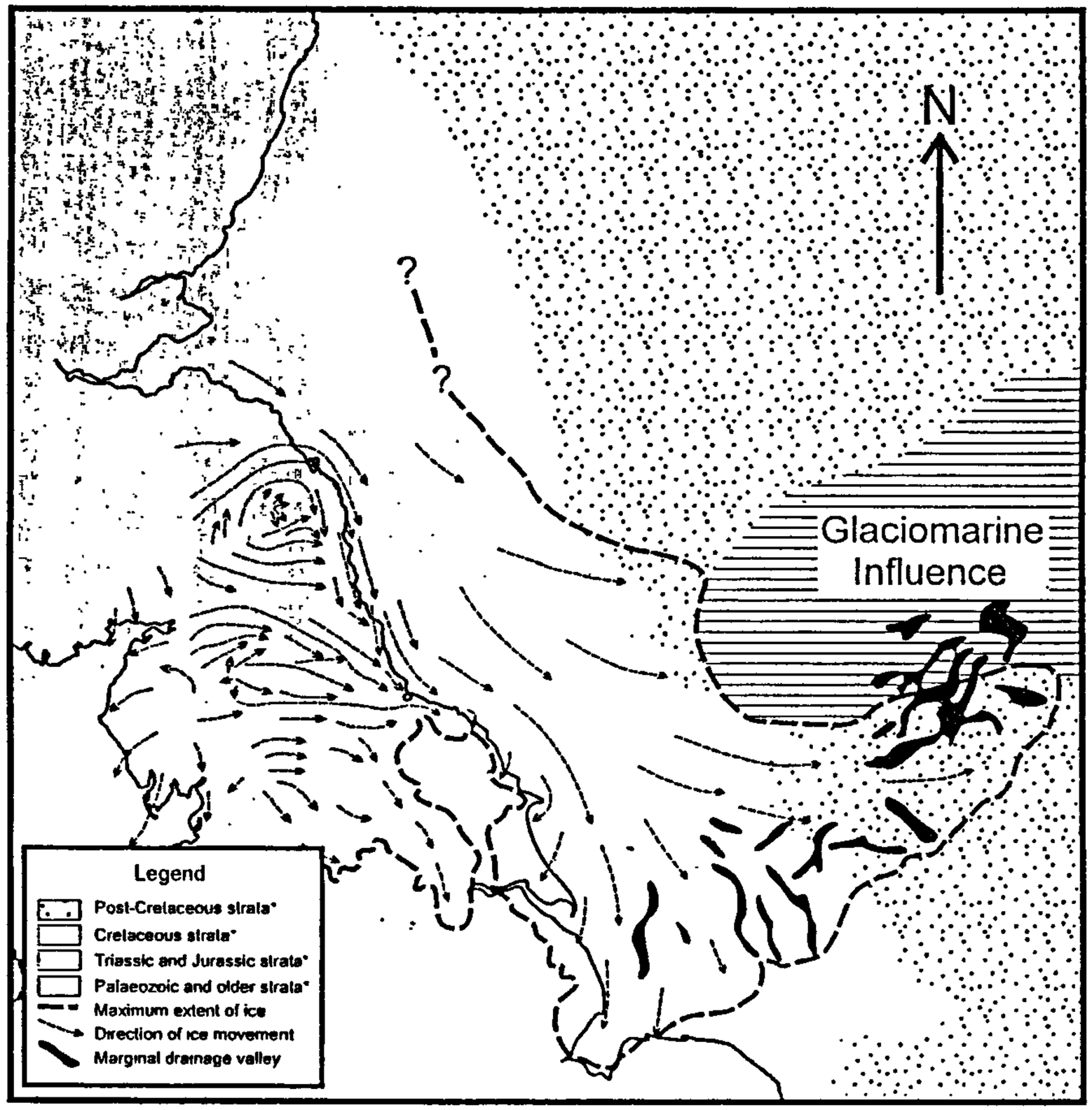

Figure 8 The Maximum Extent of the North Sea Lobe of the British Ice Sheet. Based on the work of Balson and Jeffery (1991), Catt (1991b), Lambeck (1993a) and Raistrick (1931). Geological outcrops are printed with the permission of the Director, British Geological Survey. NERC. All rights reserved.

North of Blyth, in Northumberland, the sediments found along the coast appear to have originated in the Cheviot Hills and Southern Uplands, which would allow for a simple model of southward moving ice. The presence of Cretaceous chalk fragments, however, would indicate a more complex history. Chalk only outcrops a considerable distance offshore of Northumberand. Erratics containing Chalk and chalk flints were reported by Trechman (1915) between Trow Rocks and Marsden Bay, near South Shields (NZ 385667), and by Smythe (1912) on the Northumberland coast near Newbiggin-by-the-Sea (NZ 310872). For these offshore sediments to be found this far west, the coastal ice lobe must have had a westerly vector. Figure 8 shows the outcrops of Cretaceous and older strata in north-east England. Speculation as to why the ice lobe moved parallel to the coast or at times onshore, but NOT down the present day topographic gradient eastwards, continues. Theories include:

1. The Coastal ice felt the influence of a Scandinavian ice sheet crossing the North Sea, which deflected it southwards. This theory fails to account for the absence of ice in the Central North Sea after *c.*26 Ka BP (Sejrup *et al.*, 1994).

2. The presence of a separate offshore ice centre, which either produced ice moving southwards, or deflected ice from the west. Boulton *et al.* (1977) rejected this hypothesis for climatological reasons.

3. The presence of a glacio-isostatic fore-bulge to the east, forming a topographic high, which in turn deflected ice southwards (Ehlers & Wingfield, 1991).

4. Glacioisostatic down-warping at the eastern margin of the British ice sheet, produced a relativly low region running parallel to the present coast, which channeled ice southwards (Ehlers & Wingfield, 1991).

5. The natural geometry of the western margin of a southwards flowing ice lobe would have given a west/south-west vector to the ice. Figure 8 illustrates that an ice lobe moving towards the south-east would actually have had an edge, as it spread near its margin, which had a westerly component to its movement.

6. The dynamics of the surging ice lobe may have been controlled by the lithologies outcropping below the ice. The pattern of outcrops of various ages of strata along the English coastline are shown in Figure 8. The ice lobe appears to have moved over and parallel to the outcrops of Triassic to Cretaceous sediments offshore. It has been speculated that lithology has an influence on drainage patterns under ice sheets. Indeed, Figure 8 shows five prominent marginal drainage valleys at the southern edge of the ice lobe over the Cretaceous outcrop. It is possible to speculate that sub-glacial drainage and deformation styles favoured rapid flow of ice to the south-east.

The model proposed by Boulton *et al.* (1977) suggested that an eastern coastal ice lobe might become unstable around the latitude of the present river Tees. The extensive coastal Quaternary sections in north-east England provide an ideal opportunity to follow a potential surge back up its length, and to test models of depositional styles. Classic deformational structures, including clast pavements, are visible in coastal sections of Northumberland, but have yet to be formally described.

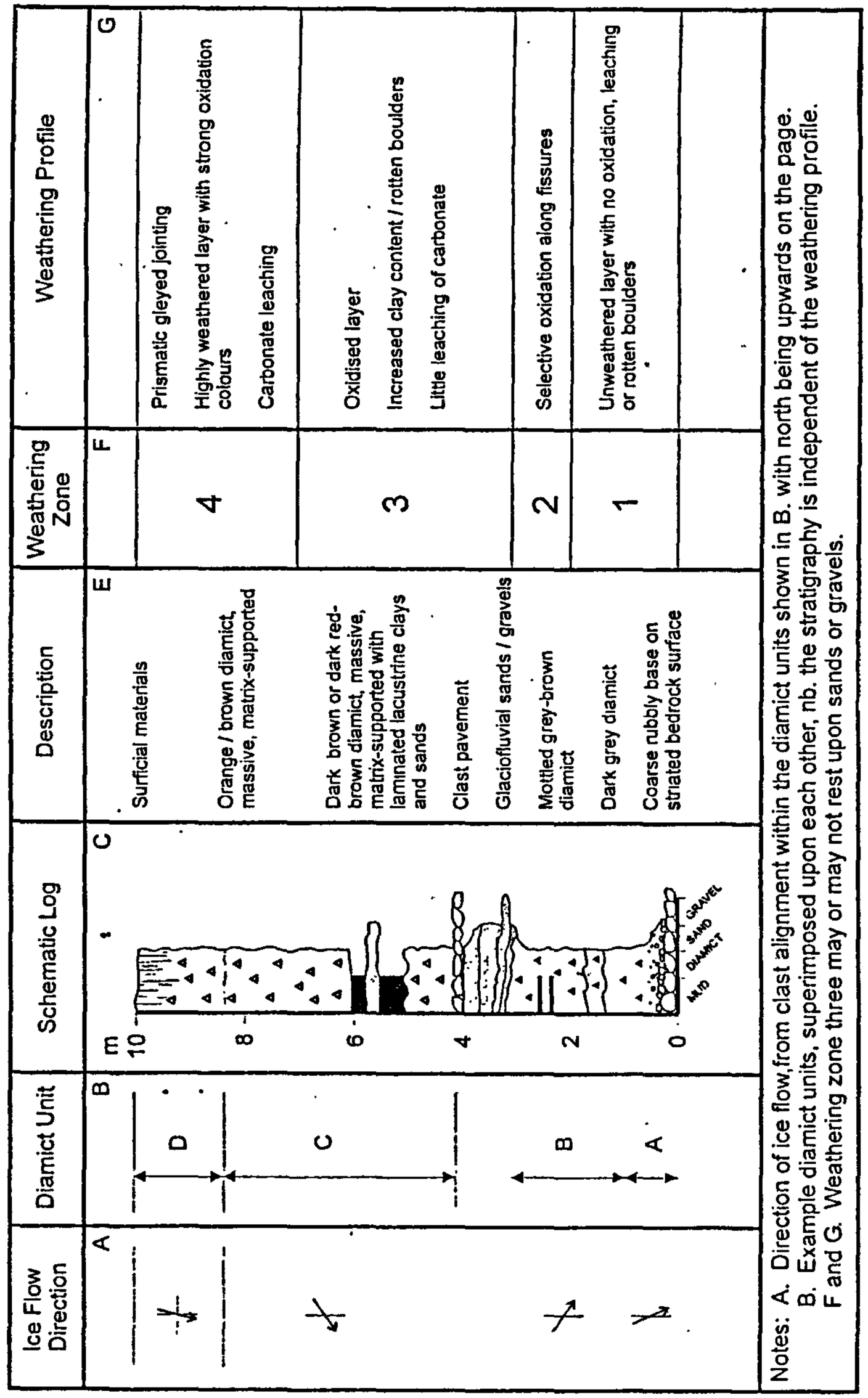

Figure 9 Generalised characteristics of a lodgement till in coastal Northumberland. After Eyles and Sladen (1981) and Eyles *et al.* (1982)

Glaciomarine Influences

To some extent the current interest in a possible glacio-marine influence on the east coast margin of the British ice sheet stems from work carried out on the Irish Sea Basin. Eyles and McCabe (1989) envisaged the Irish Sea Basin as being glacio-isostatically depressed, which led to a high relative sea-level, marine influx, instability of the ice sheet and rapid calving. The subsequent surges and rapid drainage of the ice stream might have stranded large amounts of ice in peripheral lowlands and initiated the collapse of the British Ice Sheet.

Eyles *et al.* (1994) re-examined the sedimentologicai evidence from Holderness in Yorkshire and interpreted the sequences as glacially reworked marine sediments. Their interpretations also provided support for models showing surging ice lobes along the eastern coast of England. Eyles *et al.* (1994) recognised the difficulties in explaining why ice surged in lobes down the eastern coast, but speculated that a build up of ice thickness in source areas; substrate water pressures and the overriding of fine grained preglacial muds may have been significant. Indeed extensive proglacial bodies of water seem to be a feature of much of the glaciation of north-east England (e.g. Lake Humber, Lake Pickering and Glacial Lake Wear). Eyles *et al.* (1994) went further to speculate upon the role of high relative sea levels in controlling the creation of these large bodies of water.

Offshore studies in the southern North Sea (Balson & Jeffery, 1991) show what have been interpreted as the distal equivalents of the deposits outcropping at Holderness and Hunstanton in eastern England. These deposits, the Bolders Bank Formation, are Devensian clay-ridge diamictons interpreted as lodgement till. To the east and nort-east they pass laterally into the Dogger Bank Formation, which contains a dinoflagellate flora indicating shallow open marine waters (Balson & Jeffery, 1991). There is therefore the possibility that marine influences were close, both in time and space, to the eastern coastal ice lobes. Conclusive evidence has, however, proved hard to find. Future research in the north-east of England will need to address the possibility of dinoflagellate records in preglacial bodies and sedimentological evidence onshore.

Problems of Interpretation

The classic interpretation of the glacial sediments in the north-east England uses a tripartite stratigraphy. The lowest sediments in this sequence are usually grey/blue over-consolidated diamictons. interpreted as lodgement tills. Next are glaciofluvial sand and gravel bodies. Red/brown diamictons, again interpreted as lodgement tills, finish the sequence (Hughes *et al.*. 1998; Figure 9). Despite many decades of investigation, this simple interpretation remained in vogue. both in Yorkshire and north-east England, and was the basis for long-distance correlation. Eventually the classic interpretations based on colour were attacked on various fronts. Catt (1991a,1991b) and Madgett and Catt (1978) reinterpreted the Hessle Till at the top of the Yorkshire sequence as resulting from post-glacial deep weathering. Eyles and Sladen (1981) continued this work in Northumberland, by showing that this profile resulted from the weathering of a single Late Devensian till sheet complex. Weathering of sulphide-rich minerals in the upper layer of the diamict sequence stained it

from a fresh grey colour to orange-brown. Eyles *et al.* (1982) recognised that there was more than one lodgement till unit present at most sites investigated, and that they were stacked either conformably or unconformably upon each other. The final weathering profile seen in an exposure overprinted this separate sequence of diamict units, and meant that interpretations based on colour were fundamentally flawed (Figure 9). Hughes *et al.* (1998) provided much evidence for the stratigraphic succession from exposures at opencast coal sites operated within the region.

Work by McCabe *et al.* (1998) has cast doubt on whether the glacial history of north-east England effectively ended at the end of the Dimlington Stadial, by suggesting that a major ice readvance occurred about 14,7-14 ^{14}C ka BP, associated with the North Atlantic Heinrich Event 1. The assumption that on the broadest scale the vast majority of glacial sediments in the north-east England correlated directly with those of the Dimlington Stadial in Yorkshire, has been questioned. This assumption implied that a complex sequence of sediments had been deposited in Yorkshire during a relatively short period of time, associated with a Late Devensian maximum occurring shortly after 18 ^{14}C ka BP. The suggested interpretation of McCabe *et al.* (1998) would allow much more time for glacial and periglacial activety to occur.

Conclusions

There is a continuing, and perhaps growing, need to 'ground truth' theories based on modern glacial analogues against the sediments produced by palaeo-ice sheets. That so little current research is being undertaken in the north-east England into these sediments raises questions about how far the pendulum has swung away from attempts to understand regional Quaternary geology. If the glacial geology of Yorkshire remains poorly constrained and therefore controversial, that further north in Durham and Northumberland is even more so. Opportunities abound for research into fundamental questions about deposits that cover 70% of the north-east, but in many ways are as enigmatic as they were 130 years ago when research began. The work of McCabe *et al.* (1998) reminds us how very little is really understood about the glacial history of north-east England. In particular, we have no dates with which we may constrain our models, and models that contradict each other.

Acknowledgements

Figure 8 is in part based on the following BGS maps:
(i) Geology of the UK, Ireland and Continental Shelf - South Sheet 1:1 000 000
(ii) Geology of the UK, Ireland and Continental Shelf - North Sheet 1:1 000 000.
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PAPER B3 TILLS IN EARTHWORKS

Clarke, B.G., Hughes, D.B. and Hashemi, S. 2002.

Characteristic Parameters of Tills in Relation to Earthworks.

Conference in Dublin, 2002.

Characteristic Parameters of Tills in Relation to Earthworks

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University of Newcastle upon Tyne

ABSTRACT Evidence from fresh exposures in subglacial tills shows that they contain lenses and layers of sands and gravels and softer clays. This suggests that many subglacial tills could be described as deformation tills and excavation failures should be expected. Foundations in glacial tills are often over designed. Extensive testing programmes have shown that the mobilised strength and stiffness of these tills are underestimated. Further, the design methods and chosen parameters may not be appropriate for the subglacial tills. These issues need to be addressed if economic, safe solutions are to be developed for earthworks in glacial tills.

1 INTRODUCTION

Glacial till is one of the most variable of engineering soils. It is defined as a poorly sorted mixture of clay, sand, gravel, cobble and boulder sized material deposited from glacier ice (after Hambrey, 1994). The majority of tills in the UK are subglacial tills, that is they are tills deposited at the base of a glacier. They often contain waterlain deposits (i.e. glacifluvial and glacilacustrine deposits). This generic description of tills can be applied to Irish tills (Hanrahan, 1977) and, in particular, to tills in the Dublin area (e.g. Farrell and Wall, 1990) which are also considered to be subglacial tills.

In soil mechanics terms, subglacial till is a non textbook material, in that it is characteristically neither sand nor clay. Irish subglacial tills, for example, are uniformly graded (Hanrahan, 1977) and contain gravel, sand, silt and clay. Such tills do not conform to the depositional models upon which much of soil mechanics theory is based (Clarke et al, 1997). Further, it is often difficult to measure the properties of these tills because of the difficulties of sampling and testing clays with gravel particles and softer inclusions.

A lack of appreciation of the effects of depositional and post depositional processes on till properties makes it difficult to understand the relationship between measured properties and predicted behaviour. This has led to failures, claims and uneconomic designs. Thus, the selection of engineering design parameters for till should include a study of the geological history, the intrinsic properties and the in situ characteristics of the material (Clarke et al, 1998) if economic, safe designs are to be produced. This has an impact on the Specification for Earthworks (2000).

2 THE SPECIFICATION FOR EARTHWORKS

The tills shown in Figure 1 fall into Class 2C of Table 6/2 of the Specification since they are a stony cohesive material that could be used as general fill. This Class can contain any material or combination of materials.

The material properties required for acceptability are grading, water content, Moisture Condition Value and undrained shear strength. The Atterburg Limits, shown in Figure 2, are acceptable since the liquid limit is less than 80% and the plasticity index is less than 55%. The undrained shear strength has to be determined from tests on remoulded soil which is compacted, at its natural water content, into a 100 mm diameter mould. The tests have to be carried out at 200 kPa cell pressure. It does refer to the possibility of measuring c and ϕ which are assumed to be 'undrained' parameters. Very few guidelines are given regarding construction in or with till other than the need to be aware of cobbles and boulders. There is no mention of the effects of inclusions. They have to be taken into account when choosing the thickness of layers to construct an embankment.

3 GLACIAL TILL ORIGIN

The types of glacial tills of Ireland are described by Hanrahan (1977) as

- coarse-grained deposits often found as water sorted melt out features
- 'head', a superficial deposit, which may be a supraglacial deposit or a result of solifluxion

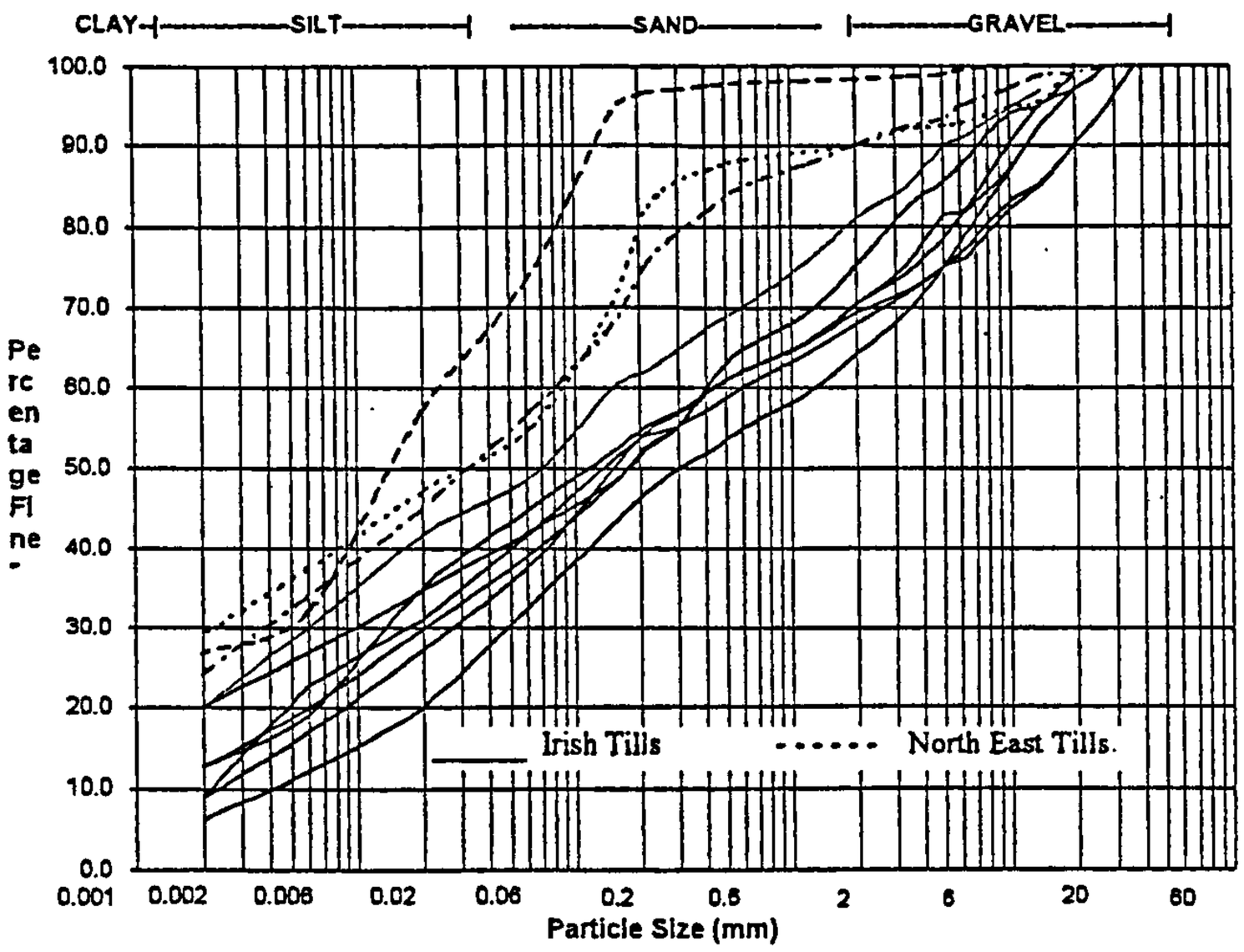

Figure 1 Particle size distribution of Irish and North East England tills

- lodgement till, a subglacial deposit
- englacial till
- drumlin till
- water sorted tills, which can be either proglacial ablation tills found at the boundary of a melting ice sheet, or water laid tills, which were formed in glacial lakes.

These tills can be grouped into two categories for engineering purposes:- subglacial tills such as lodgement till which are deposited on land, and waterlain deposits. The latter, which include laminated clays and sands and gravels, can be characterised by classic soil mechanics theory and hence dealt with by the Specification. The sub glacial tills, however, may not. There is no reference in the Hanrahan paper to deformation till.

Most of the published information on subglacial tills comes from the Dublin area. It is described as a very stiff brown or black sandy gravely clay which contains boulders and lenses of sand and gravel. The strength of the till varies from stiff (brown) to very stiff to hard (black) and is generally described as over-consolidated. This sequence has been described as a single lodgement till deposited during the last ice age with an upper, weathered layer. The upper three metres are typically brown, though some brown till is found at lower levels. It also has been described as two separate deposits of lodgement till, a lower black till and an upper brown till. While it is described as clay, there are very few true clay type particles within the matrix. As with other subglacial tills, the finer particles are predominantly rock flour.

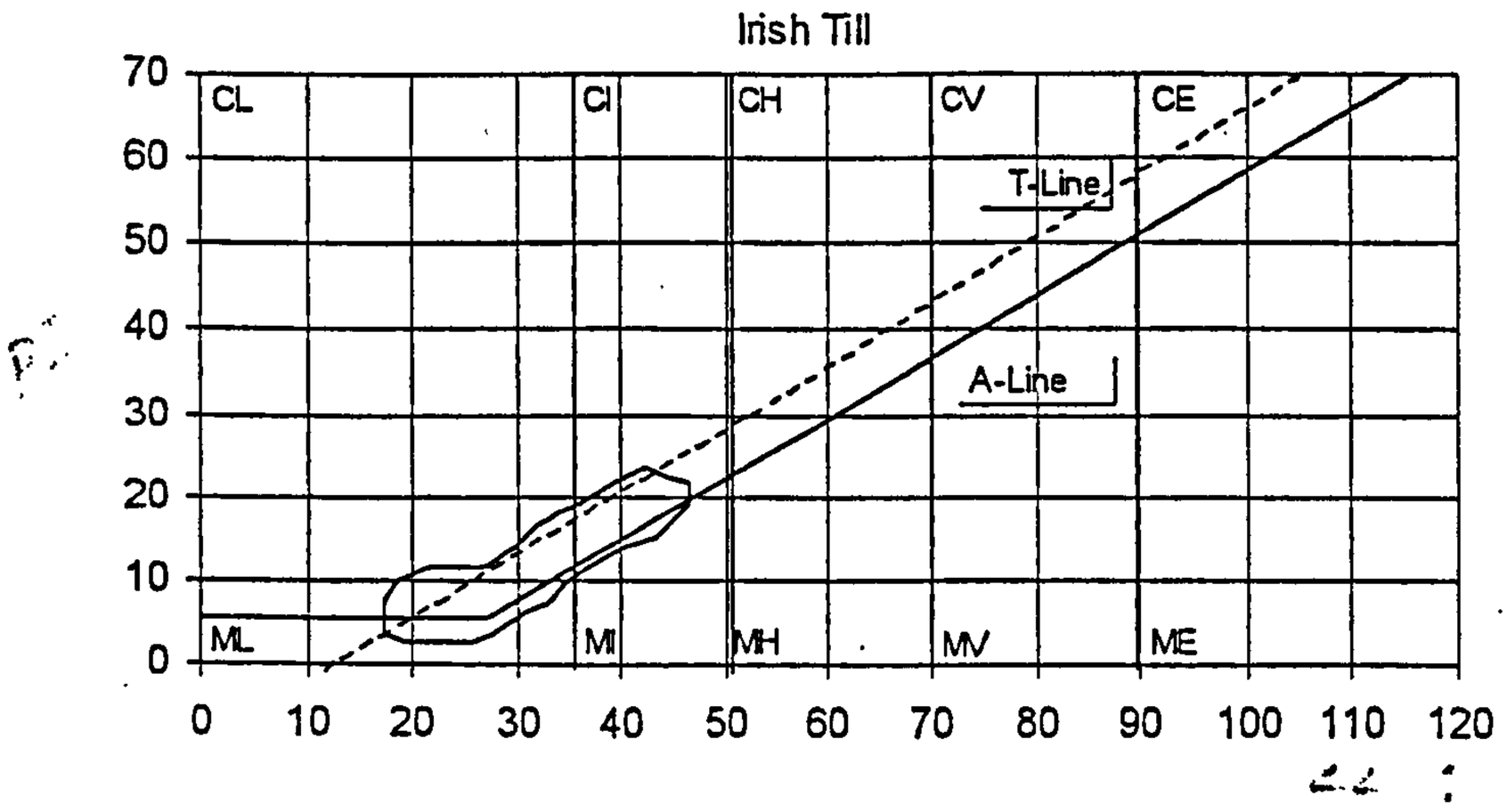

Figure 2 Atterberg Limits for Irish tills

Hughes et al (1998) have completed an extensive study of the tills of the Northern Counties of England. This arose because, like Ireland, the majority of the engineering soils were glacial in origin, and because they had access to an extensive database from investigations for opencast coal mines throughout the area. The tills in the English Northern Counties are different from the Irish tills but there are common features, which suggest that some of their findings could be assigned to the Irish tills.

between the Atterberg Limits of the Irish and Northern England tills. Both layers of the English tills contain lenses of sand and gravel, and laminated clay, although these are more commonly found in the upper layer.

The lower till has been described as a lodgement till. The upper till has been described as either a weathered grey till or a separate lodgement till. Hughes et al (1998), however, are of the

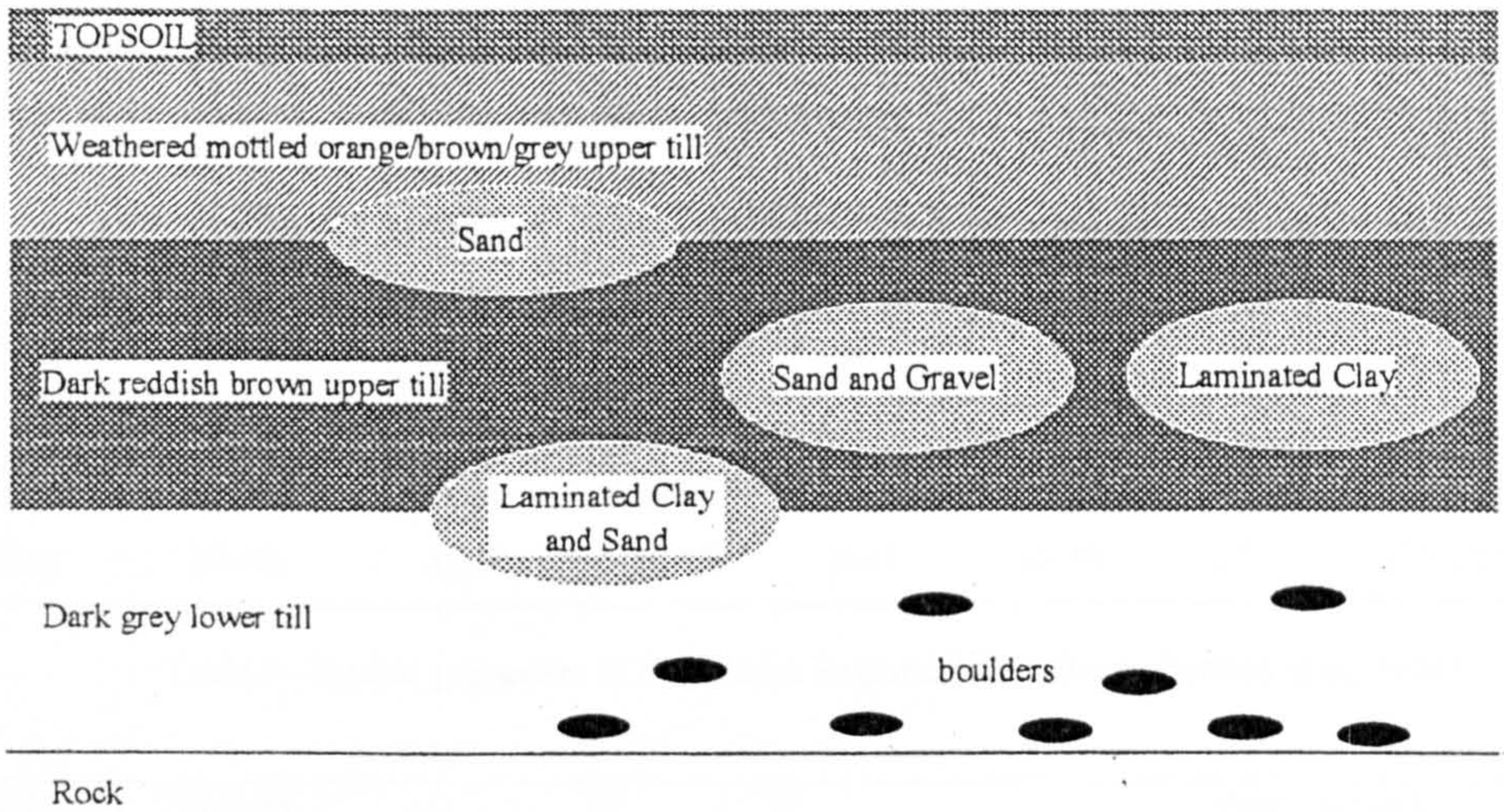

Figure 3 A simplified schematic diagram of the tripartite succession of the tills in Northern England (after Robertson et al, 1994)

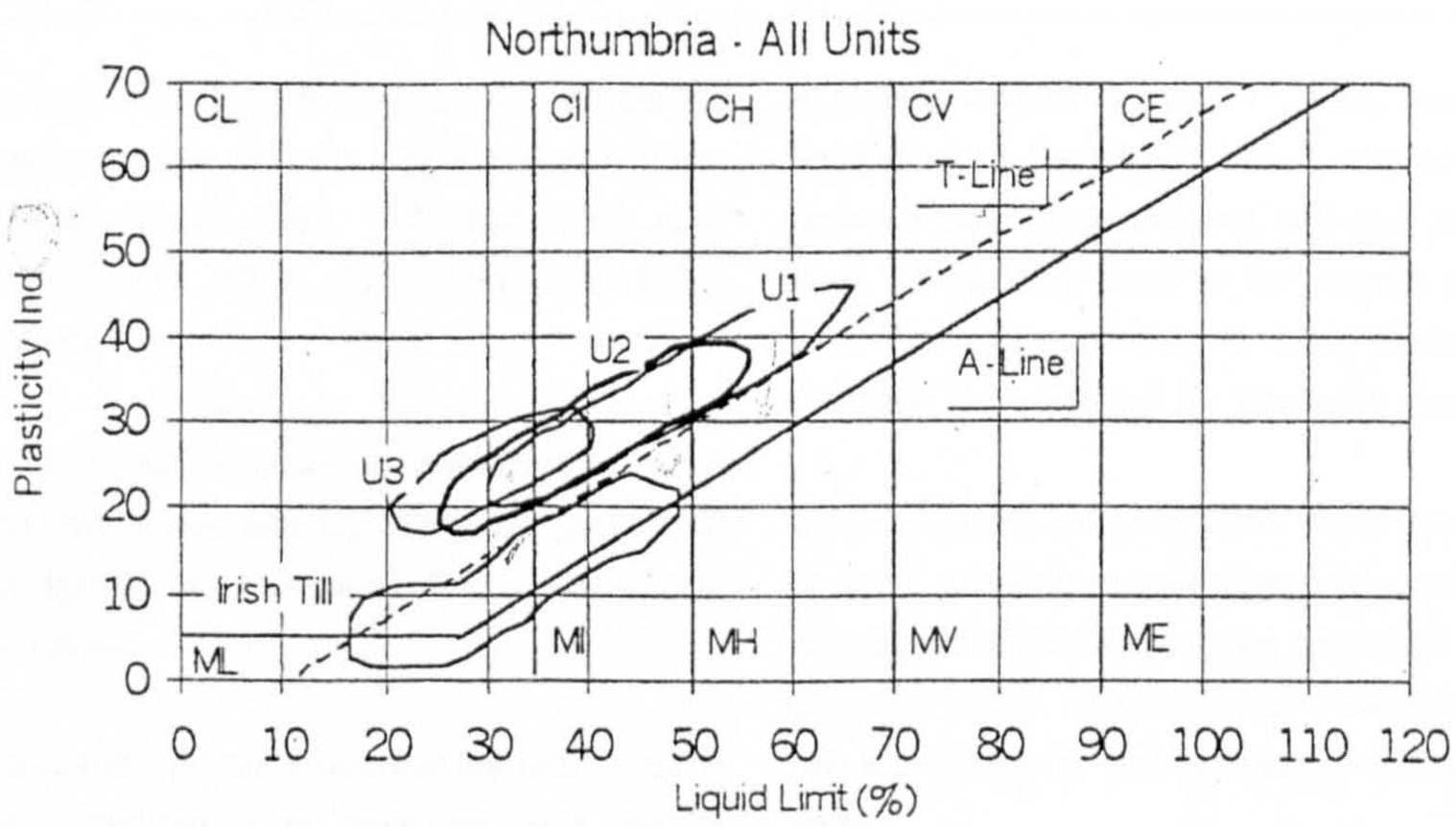

Figure 4 A comparison between the Atterberg Limits for the Irish Tills and tills from the North of England (U1, U2, and U3 refer to the units of till with U1 at the top and U3 at the base)

In the North East of England the tills often comprise two layers, separated by a layer of either laminated clay or sand and gravel as shown in Figure 3. The upper layer varies from red/brown to brown in colour; the lower layer is dark grey or grey/brown. Both layers can be described as sandy silty clay containing gravel and occasional cobbles and boulders. Usually, only the red/brown till occurs where the overall thickness to rockhead is less than about 8 m. The stone content is less than that of the Irish tills as shown in Figure 1. Figure 4 shows a comparison

opinion, based on extensive photographic evidence of various fresh features within the tills, that they could both be deformation tills (as defined by Benn and Evans, 1998) with the near surface layers being weathered. The deposition processes of lodgement and deformation tills are similar, that is they are spread into place beneath moving ice. The difference lies in the amount of comminution, which results from the amount of shearing and, hence, distance moved. Lodgement tills have no relics of the source material, whereas deformation tills do. The presence of localised lenses of brown till and waterlain deposits

within a very stiff to hard till suggests that such tills cannot be lodgement tills. Indeed, they could be deformation tills formed of previously deposited melt out, englacial and subglacial tills. This is not unexpected because the ice would have advanced several times during the Quaternary period, especially during the Anglian and Late Devensian glaciations. Each ice advance transported till which had been deposited during the previous advance. Given the fact that tills with such inclusions occur over a wide area suggests that deformation tills are common. Also, the presence of extensive layers (sometimes several kilometers wide) of sand and gravel and/or laminated clay between the upper and lower tills indicates the former existence of pro-glacial lakes, as these are glacilacustrine deposits. This suggests either two successive ice sheets (Smith, 1981, 1982 and 1994) or a two-tiered ice sheet (Carruthers, 1953; Catt (1991 (a) and (b)) during the Late Devensian.

This conclusion has important engineering implications. The possibility of encountering water bearing sand and gravels or weaker layers of clay should be anticipated, though their exact location will never be known.

It is often assumed that the tills are overconsolidated. This is based on the assumption that the in situ strengths could only be achieved through a process of consolidation due to the weight of the advancing ice and swelling as the ice melted according to classic soil mechanics theory. However, the till was not only gravitationally deposited, it was spread into place through a process of pressure and shear. Thus, the deposition processes produced a very dense material but not only through gravitational compaction. At any particular location, several different processes could have taken place due to successive periods of glaciation and interstadial flow. Thus, a present day till may have been moved several times and have been subject to a variety of pressure regimes.

Table 1 Pore pressure regimes within till during deposition

Base of Glacier	Base of Till	Underlying Rock	Pore Pressure
frozen	unfrozen	aquifer	no excess
frozen	unfrozen	no aquifer	excess
unfrozen	unfrozen	aquifer	no excess
unfrozen	unfrozen	no aquifer	excess
frozen	frozen	not relevant	none

Table 2 Typical properties of Dublin Tills (after Farrell and Wall, 1990 and Farrell, 1989)

Till	% fines	LL %	PI %	W %	N	ϕ'	shear strength kPa
Brown	35		10-15		15-20	35^0	150-250
Black	24-46	19-31	10-15	8-11	35-40	37^0	150-250

Table 3 Typical properties of North East England Tills (after Robertson et al, 1994)

Till	% fines	LL	PI	W	γ Mg/m^3	shear strength kPa	ϕ'
upper	39		9-39	9-34	1.62–1.93	50-410	27-35
lower	23		8-27	9-23	1.76-2.00	65-410	32-37

A number of pore pressure regimes, Table 1, could have existed depending on the weight of ice, the pressure applied, the temperature and the underlying soil and rock strata (Boulton and Paul, 1976). It is possible that excess pore pressures generated as the ice advanced did not dissipate until the ice retreated. In that case, the till would be 'normally consolidated'.

In conclusion, till may have been gravitationally compacted, sheared, possibly reworked and weathered. Unlike most engineering soils, till has not been deposited through water. The implication is that the concept of consolidation, on which much of our knowledge of soil mechanics theory is based, may not be valid.

4 GLACIAL TILL PROPERTIES

Typical properties of Dublin Tills are given in Table 2. Table 3 gives similar data for Northern England tills.

Boulton and Paul (1976) suggested that tills can be characterised by the fact that they exist on the T-line as shown in Figure 2. The denser, stiffer tills tend to be grouped at the lower end of the T-line. The weathered and upper tills cluster further up the line. This indicates that the fine grained component of the tills behave

in a clay like manner with low sensitivity even though they contain very few clay particles.

Clay dominant tills, such as those in Northern England, can be sampled by driven tubes. Thus, it is possible to carry out triaxial tests on 'undisturbed' specimens. Published data on the Dublin tills suggest that this is not possible in Ireland because of the stone content. Therefore, the strength is either derived from correlations with N, the Standard Penetration Test blow count, or measured in tests on remoulded specimens. It is often assumed that clay tills are insensitive in that the remoulded strength is similar to the in situ undrained shear strength.

Rotary coring has been carried out in Northern England, where very hard and very stony (lower) tills are encountered. This technique may present a way of obtaining relatively undisturbed samples for strength and deformation testing of the Irish tills.

It is difficult to obtain undisturbed specimens of Irish subglacial till, therefore undrained strengths are estimated from correlations between SPT blow counts and tests on remoulded specimens. A value of 6 for Irish tells is commonly used. Stroud (1975) presented a correlation between undrained shear strength and blow count based on plasticity index. This correlation is for fissured clays which suggests that the correlation is too low in unfissured tills. Remoulded specimens will be partially saturated hence an 'undrained' test will be partially drained though the amount of drainage is unknown. The strength will increase as the amount of drainage increases. The effects of density and drainage may cancel each other out but the contribution each make is unknown. If density is the dominant effect, it implies that the SPT correlation for such tills could be too small.

The data used by Stroud, which covered plasticity indices between 15% and 65%, showed that the ratio varies between 7 and 4.1. Orr (1993) suggests, for one site, that the ratio for Dublin tills is 6. This was confirmed by Farrell and Wall (1990) for the brown till. Hanrahan (1977) suggests it varies between 4.5 and 6. However, these correlations are based on remoulded strengths. Figure 5 shows data from a site in the Northern England in which results of triaxial on undisturbed specimens are plotted against depth together with values of strength derived from SPT N values. It shows that the correlation could be as high as 9. There is evidence from piling contractors that quoted correlations between undrained shear strength and blow count are too low. This suggests that earthwork calculations should be based on a factor of 6 since the till is remoulded whereas foundation design calculation could be based on 9 since the in situ strength is mobilised.

Albeit that the till is insensitive, an effect of remoulding is to reduce its density. Subglacial tills are very dense due to the method of deposition. It is not possible to recreate those densities in the laboratory through a process of conventional compaction or consolidation. Strength will reduce as the density reduces, thus results of tests on remoulded soils are probably less than in situ strengths

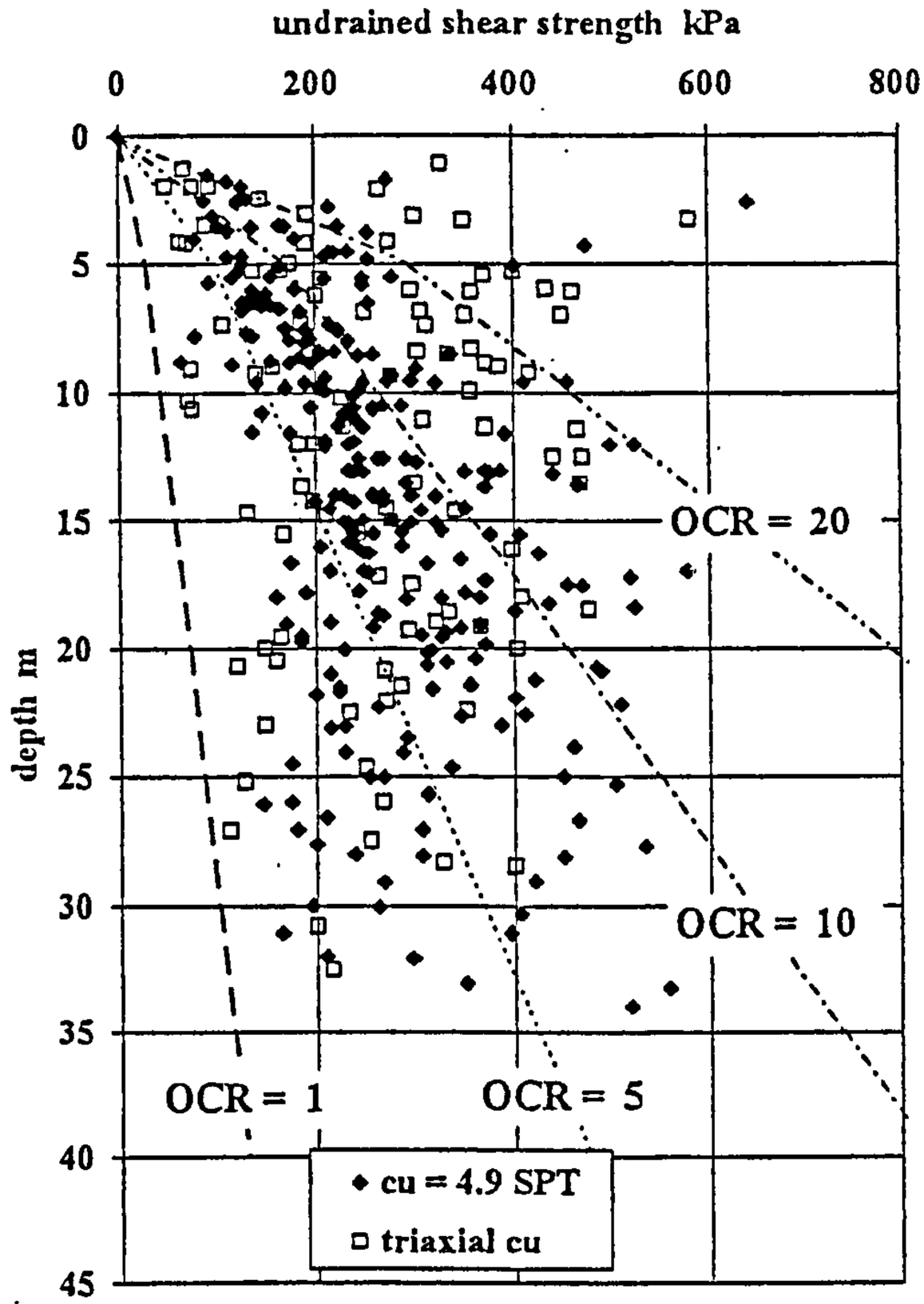

Figure 5 Undrained shear strength derived from Standard Penetration and triaxial tests

Although the Irish tills are described as clay tills, typical angles of shearing resistance of 37^0 suggest a sand type material. The Irish tills are described as clay dominant tills and, given their behaviour, are treated as fine grained soils but with characteristics of coarse grained soils. Interestingly the N values for Irish tills relate to an angle of shearing resistance in excess of 37^0 which confirms the suggestion that in situ strengths exceed laboratory measured strengths in tests on remoulded soils.

A typical value of angle of shearing resistance for Irish till is 37^0. This is obtained from tests on remoulded specimens. Figure 6 shows a comparison between effective stress tests on remoulded and natural tills from Northern England. It is clear that remoulding reduces the strength; a fact which could be related to a reduction in density. Clarke et al (1997) have suggested a technique based on Burland's (1990) intrinsic compression line, to predict in situ values of strength based on tests on remoulded

specimens. This allows design parameters to be obtained from samples of poor quality.

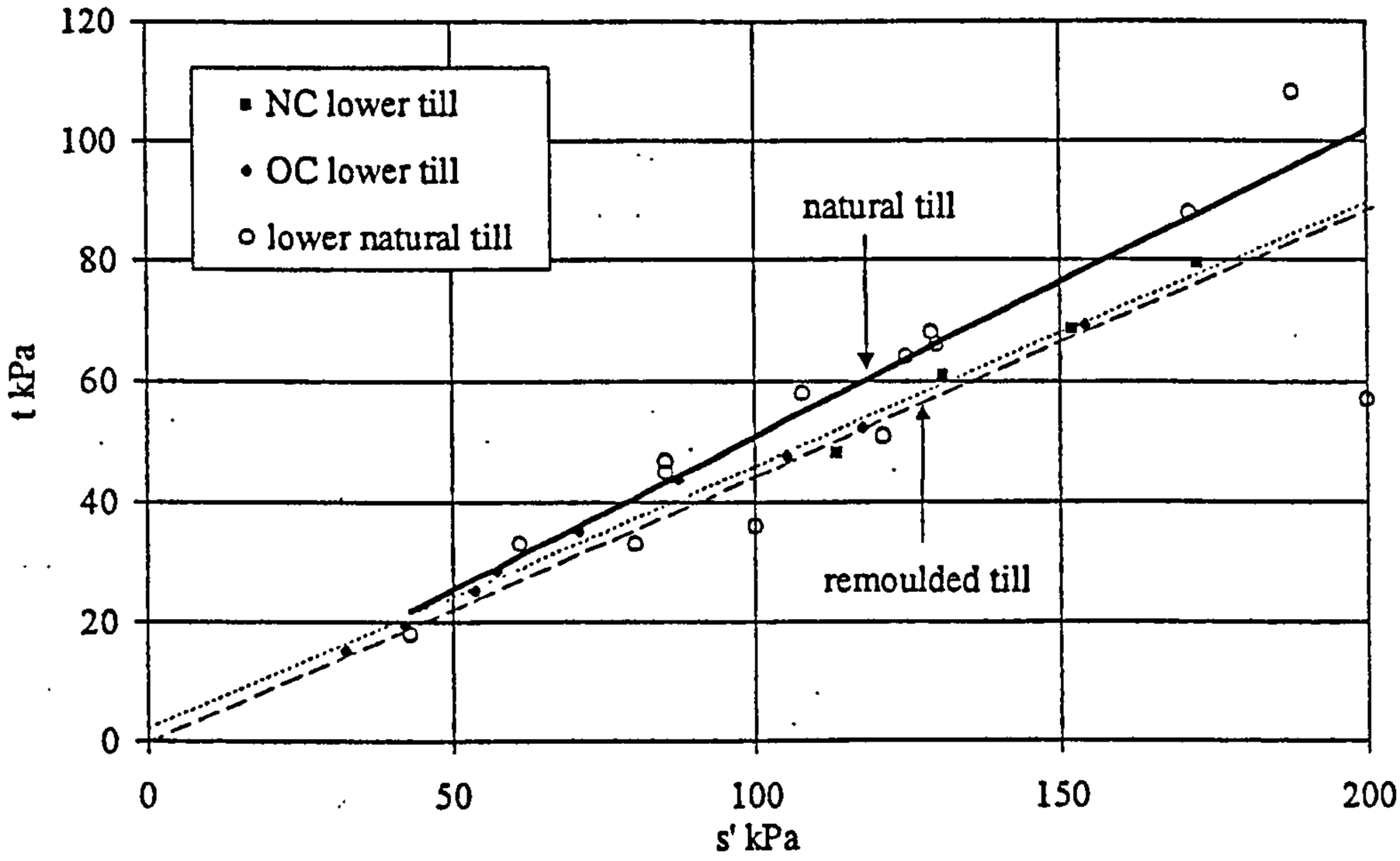

Figure 6 Comparison of effective strength parameters from tests on 'undisturbed' and remoulded till form Northern England

It is often assumed that construction loading in clay produces positive excess pore pressures, which dissipate with time leading to settlement. The amount of excess pore pressure generated is a function of the stiffness of the soil; the speed of dissipation is a function of the coefficient of consolidation.

Farrell (1989) suggested the coefficient of volume compressibility for Irish tills varies between 0.03 m^2/MN and 0.06 m^2/MN. Tomlinson (1995) suggests that tills could be considered soils of low to very low compressibility with the coefficient of volume compressibility less than 0.1 m^2/MN. The generation of pore pressure during loading is a function of the stiffness of the soil; the stiffer the soil the lower the excess pore pressure generated. The dissipation of excess pore pressure is a function of the permeability and the stiffness of the till. Due to their density and particle size distribution, tills have a low permeability. 10^{-10} m/s is a typical value. This implies tills are treated as impermeable materials and hence are suitable as containment material. A word of caution is necessary, however, when considering natural tills because tills can contain lenses of more permeable material.

The time it takes for excess pore pressure to dissipate can be expressed in terms of the coefficient of consolidation. Farrell (1989) suggests that this coefficient exceeds 70 m^2/yr for Irish tills, which is typical of a coarse grained soil. This implies that any excess pore pressure generated will dissipate rapidly. Thus, the black till could be considered to exhibit drained behaviour during loading even though it is a clay dominant soil. This is consistent with observations in other subglacial tills and may account for the fact that actual pile capacities are known to exceed design capacities based on undrained shear strength.

Looby et al (1995) undertook an instrumented pile test in Dublin. They showed that pore pressures during driving were low or negative. These increased when driving stopped but excess pore pressures had dissipated within one week. Pore pressures on loading reduced. These observations support the statements given above. The undrained shear strength was estimated to be 450 kPa based on SPT N values of 65 to 80. Back analysis of the pile test suggested that the capacity was between two and four times greater than the design value. This is consistent with experience of piling contractors in the UK. 60% of the capacity was developed at the end of the pile, which is typical of piles in coarse grained, free draining soils. The evidence from this test supports the view that pile designs in these soils should be based on effective strength parameters rather than undrained shear strengths. The base capacity derived from effective strength parameters is 460 kN; whereas it is only 240 kN when derived from undrained shear strength.

5 APPLICATION IN EARTHWORKS

Irish subglacial tills are classed as stony cohesive fills yet there is evidence that, in some circumstances, they behave as coarse grained soils. It would appear that the tills can be considered cohesive because they are able to retain a shape without external support. For example, a vertical face will remain standing. For that reason, the undrained shear strength may be used in calculations for short term stability when tills are used as fill material.

Alternatively, stability calculations can be based on effective strength parameters. The Specification suggests that these can be derived from shear box or triaxial tests. No test details are given for stony cohesive fill. It would seem appropriate to use as large a specimen as possible, perhaps 300 mm shear box or 100 mm triaxial specimens, to ensure that the particle size distribution is representative. Specimens are likely to be remoulded because of the difficulty of sampling. Tests should be carried out on saturated specimens compacted at the natural water content to ensure that the worst case conditions are measured. The values of strength from these tests are likely to be less than the in situ strength because of the difference in density. Thus, conservative values are obtained. It should be noted that Vaughan (1994) observed that natural slopes in tills in the North of England were typically 45^0, whereas laboratory measured values of angles of shearing resistance for tills in that area are typically 36^0. A geomorphological study of natural slopes on Irish subglacial tills may provide interesting data.

The Specification suggests that 'undrained' shear strength parameters, c_u and ϕ_u, can be derived from undrained tests. These, commonly quoted in site investigation reports, are derived using a curve fitting method. They are not intrinsic properties. It is unsafe to use c_u and ϕ_u in stability calculations because they are apparent strength parameters which will change with time and degree of saturation. It is conservative to use the c_u component only since it is less than the undrained strength of the specimen. For these reasons, it is recommended that these parameters should not be considered.

Fill slopes can be designed on effective strength parameters based on tests on remoulded specimens. This will give adequate slopes. Excavated slopes, however, have to be treated with care because of the presence of lenses of weaker and free draining, water bearing soils. Many failures in excavated slopes in tills can be attributed to these lenses. This suggests a more detailed investigation may be necessary at such locations to locate these lenses.

6 CONCLUSIONS

The Irish tills, particularly in the Dublin area, can be described as clay matrix dominant tills but their behaviour in some situations may be similar to a free draining material. This is consistent with observations of UK subglacial tills.

Correlations between in situ undrained shear strength and Standard Penetration Test results are possibly too low since they are based on tests on remoulded specimens, i.e. specimens which are less dense than those in situ. Using current correlations may produce conservative designs unless they are applied to remoulded till.

Fill slopes designed using empirically derived undrained shear strengths or effective strength parameters derived from tests on remoulded specimens will be adequate. Excavated slopes will be conservative because all evidence suggests that in situ densities are greater than remoulded densities. However, excavated slopes may contain lenses of weaker soils which would dominate any failure mechanism.

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PAPER B4 NETDATA

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NETDATA – a relational database for geotechnical properties of Northern England glacial tills.

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NETDATA - a relational database for geotechnical properties of Northern England glacial tills

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Abstract

Geotechnical investigations have been carried out for many years in order to identify the depth, extent and the properties of the glacial deposits of Northern England. This valuable source of data exists in various forms because of the differences in the design of the investigations, the codes of practice relevant at the time of investigations, and contractors practice. An electronic store of these data would be a valuable asset not only to future development but to help in scientific studies of glacial till. This paper presents an overview of NETDATA (Northern England Till DATA), a relational database which is designed to store that data in a consistent format, allowing flexibility for further use. The database is an independent and cost-effective system for storing and managing data, which aims to ensure that the data are coherent and consistent, as they are captured and processed. These data are maintained in a secure and controlled environment, so that they remain accessible for use as and when required but cannot be altered in any way. The design of the database and the way it operates are reviewed in this paper.

Introduction

The landscape of Northern England has been subjected to glacial activities during the last 2 to 3 million years (the Quaternary period) and as a result glacial materials of considerable thickness have been deposited in the lower lying coastal and valley areas (Lunn, 1995). These deposits are predominantly glacial tills which have been defined as a poorly sorted mixture of clay, silt, sand, gravel, cobble and boulder sized material deposited directly from glacier ice (Hambrey, 1994).

The majority of urban and industrial development and the supporting infrastructure in Northern England has taken place and will take place in the coastal areas. Hence it is important to develop an understanding of these underlying variable deposits especially as they have frequently led to problems on civil and mining engineering projects. The problems in Northern England are common to all glaciated areas in the UK which include many major urban developments and the linking infrastructure.

In Northern England glacial deposits also overlie Coal Measures strata and these deposits are frequently associated with earthworks and slope instability problems at opencast coal mines. Geotechnical investigations have been carried out over the last thirty years in connection with mining activities (Hughes et al, 1998), providing a source of information to undertake a major study into the properties of these materials. This study is important because, despite major advances in geotechnical engineering, failures in glacial tills are still common.

During the course of ground investigations, large volumes of data are collected by different techniques and in different formats for different purposes. The data cover a variety of site conditions and are used to select various parameters. These data must be judged and analysed to achieve the best engineering solutions. The difficulty arises not only with the nature of the data in each record but with the number of records that are involved, especially in large projects (Finn and Eldred, 1987). Datasets are sometimes so large and of such variety that the information they contain is obscure.

Although a large number of ground investigations have been carried out in glacial terrains in the UK, only a fraction of the information they revealed is available to the geotechnical profession (Trenter, 1999). In the past data were being captured, processed and held in various formats, by different ground investigation companies at diverse locations. The high cost of identifying what data existed, the potential duplication of effort and the risk of data inconsistency all pointed to the need to develop a central, controlled database. The design of this database would mean that the interpretations and modelling essential to any geotechnical related assessment would be supported by a reliable and consistent data source.

This database should provide a source of information for developing geotechnical models, as it is evident that tills do not conform to conventional or text-book depositional models (eg.

waterlain deposits) (Clarke et al 1997; Hughes et al, 1998). Thus the decision to develop the Northern England Till Data (NETDATA) database arose from a need to ensure that the information resulting from ground investigations is stored, managed and retrieved in an efficient and controlled manner to allow further scientific study and a 3D image of the geological and geotechnical properties to be developed for engineering and hydrological purposes.

Geotechnical Engineers are constantly solving new problems which frequently involve the re-examination and re-interpretation of old data using new theories or new knowledge and therefore they need a reliable source of data. The availability of original data allows re-calculation without costly re-analysis. The potential lifetime of such data whether archived or active is unlimited (Lowe, 1995). Data apparently superseded by more recent information or re-interpretations may still have potential value. NETDATA aims to provide a reliable source of original data.

Source of data

A large volume of literature exists on the Quaternary (glacial) geology of England (Hughes et al, 1998), but the majority of that literature, which is often contradictory, focuses on geological aspects of glacial till. Geotechnical properties can be obtained from the many thousands of samples of till material obtained from opencast mining investigations. Extensive laboratory testing has been carried out in order to produce stratigraphic and property profiles of the glacial deposits. These investigations became mandatory following the introduction of Geotechnical Codes of Practice in the opencast mining industry (Anon 1982, 1989 - and subsequently Health and Safety Commission, 1999). Ground investigations had also been carried out prior to 1982 in order to design spoil mounds (Hughes and Clarke, 1997).

Assembling a database of the available geotechnical information would compliment and enhance the geological information. While many ground investigations have been conducted, obtaining access to the data is difficult because they are owned by many organisations. An opportunity to study an extensive data set gathered over thirty years from sites across the region arose when the authors were given access to ground investigation reports produced for the former British Coal Opencast (BCO). Most of the data used for this study were found in

hardcopies of site investigation reports from various projects although some of the most recent data were in electronic form. All of the data are from sites in Northern England including those shown in Figure 1.

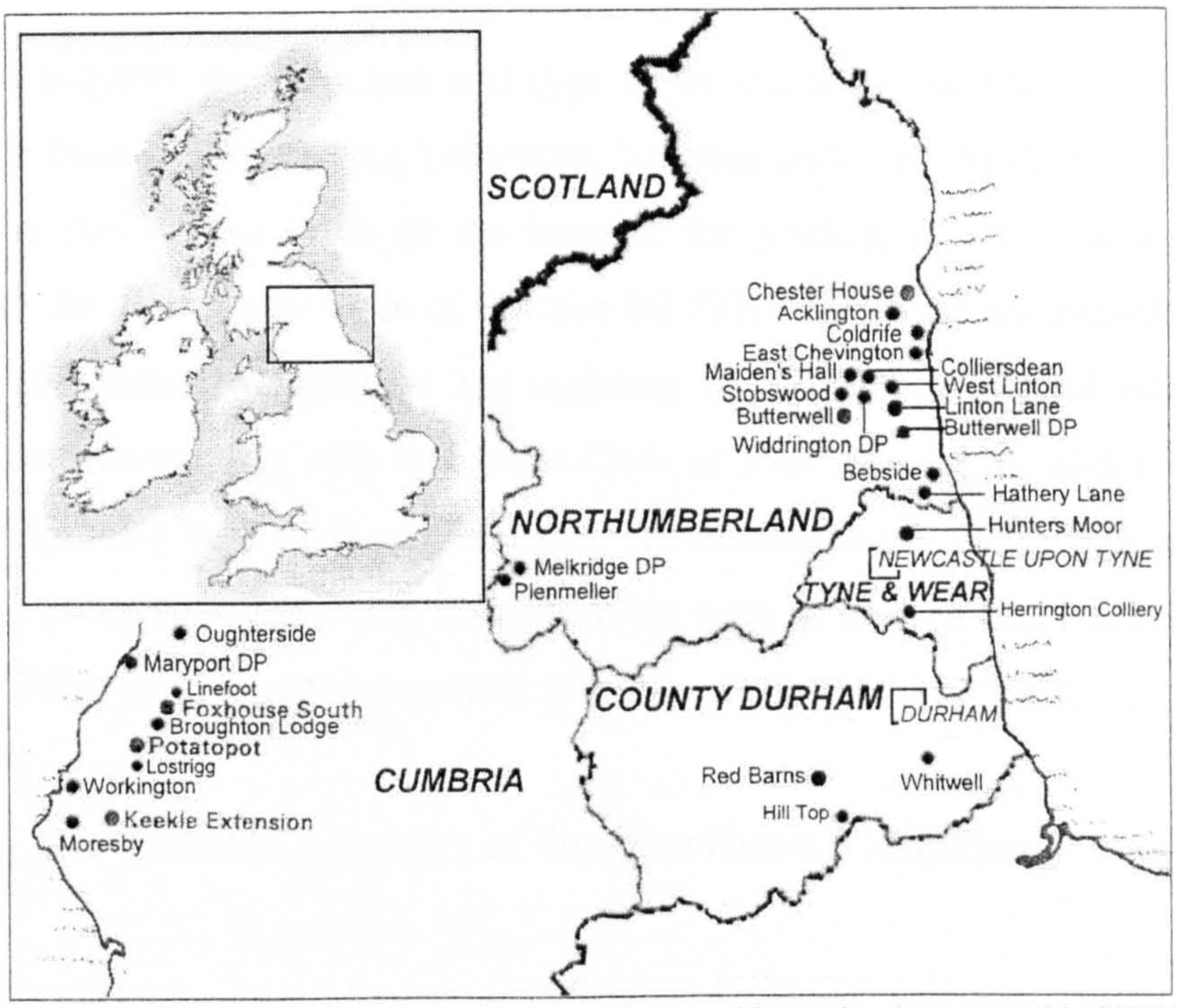

Figure 1: Location of few opencast mining related ground investigations stored in NETDATA

Opencast coal mining involves the excavation and movement of substantial quantities of material in order to extract the coal. The overburden material generally includes agricultural soil, superficial (glacial) deposits and waste rock. When mining is completed the sites are required to be restored so that the land can be used for other purposes such as agriculture and industry. Ground investigations have been carried out in order to determine the properties and extent of the superficial deposits prior to the excavation of the sites, and for other related projects such as the construction of coal disposal points or access roads. This resulted in the BCO possessing one of the largest collections of geotechnical data on glacial soils for the Northern England region. In 1994 these records were obtained under a legal agreement between BCO and the University of Newcastle upon Tyne.

Data from thirty three major site investigation projects have been used to seed NETDATA, of which eleven are from the Cumbrian coastal area, fifteen are from the North of Tyne coastal area, and seven are from the South of Tyne area (see Figure 1 for locations). The majority of

these investigations were carried out between 1980 and 1994. Hence soil descriptions and laboratory testing conform to various British Standard Codes of Practice (i.e. CP 2001:1957 and BS 5930:1981 for soil descriptions; and BS 1377:1975 and BS 1377:1990 for laboratory testing).

CP 2001:1957 requires the principal soil type to be classified as either cohesive or non-cohesive on the basis of engineering behaviour, whereas under BS 5930:1981 the distinction on the principal soil type is made on the basis of the grading (Norbury et al, 1986). Data complying with the most recent Code of Practice BS 5930:1999 were not available, but as it is the intention that constant expansion and updating of NETDATA should take place, it is intended that data complying with this latest Code of Practice will be added in the future. Before 1990 laboratory tests were carried out in accordance to BS 1377:1975, with effective strength testing procedures following standard texts such as Bishop and Henkel, (1976) and Head, (1982). These tests were subsequently introduced into BS 1377:1990.

Glacial Geology of the Northern Counties

The last major glaciation to fully cover Northern England was the Late Devensian or Dimlington Stadial ice sheet which existed from about 26,000 years to about 13,000 years ago. The Late Devensian ice sheet laid down most of the glacial deposits found in Northern England today, although some of these deposits are likely to be reworked materials from earlier glaciations (Lunn, 1995). This has resulted in considerable thickness of glacial till material being deposited in lower lying coastal and valley areas. In the coalfield areas the overall thickness of the glacial deposits is commonly 5 m to 20 m, ranging from as little as 1 m where bedrock is close to the surface, and up to 60 m or more in the pre-glacial buried channels which dissect the underlying rockhead. Frequently, where the total thickness of glacial drift exceeds about 5 to 8 m, a tripartite succession occurs in the form of a lower till, overlain by a middle sand (and gravel) or laminated clay, or both; which in turn are overlain by an upper till. The base of the upper till can vary from less than 1 m to over 20 m below the ground surface. In some locations a more complex succession with additional layers of sand and gravel and/or laminated clay is evident (Hughes et al, 1998).

The colour of the tills forming the Cumbrian coastal deposits has resulted from the colour of the source rocks which were excavated and redeposited by the ice sheet(s). Initially grey tills were deposited by advancing Lake District valley and piedmont glaciers, but then these local glaciers were incorporated into a combined Scottish - North Lake District ice sheet which deposited red till over the local grey till. The red tills predominate nearer the coast, the grey till tending to become more evident further inland. At many of the now restored Cumbrian opencast coal sites often only the lower till was recognised, which is usually grey in the eastern part of the coalfield and red in the western part (Hughes et al, 1998).

In Northumberland and Durham, the upper till is usually red or reddish brown and overlies a lower grey till. It is commonly stated that this succession was deposited as a grey lodgement till by a single ice sheet advance, and that the reddish colour of the upper till is solely due to post-glacial weathering (Eyles and Sladen, 1981; Lunn, 1995). There are, however, some aspects of the composition of the glacial succession which do not seem to be adequately explained by this single deposit and weathering concept. These include the red (upper) till generally having a much lower stone content (gravel, cobbles, boulders) than the grey (lower) till (Beaumont, 1968). Also the frequency and extent of the sand/gravel laminated clay layers which in some cases are well over a square kilometre in area, are very probably pro-glacial (glaciolacustrine/glaciofluvial) deposits and therefore indicate that the ice front was receding and releasing debris laden meltwater before the area was again overlain by later ice and ice transported deposits (Hughes et al, 1998).

In the county of Tyne and Wear an extensive glaciolacustrine deposit is often associated with difficult geotechnical engineering problems, and is referred to by Smith (1994) in his Memoir of the Sunderland area as the "Tyne and Wear Complex". Towards the end of the Late Devensian glaciation eastwards flowing meltwater issuing from retreating western ice was cut off by northern ice, so creating a series of ice-dammed lakes, the largest of which was the Glacial Lake Wear. The resulting Tyne and Wear Complex deposits are generally interbedded laminated silty clays and clayey silts, plus occasional fine grained sands, stony clays and gravels. The thickness of this formation is commonly between 5 m and 15 m, but can be up to 55 m. These deposits are highly compressible and low of shear strength.

After the Late Devensian ice melted, permafrost returned to the region during the Loch Lomond Stadial (ie 10000 years ago). This resulted in solifluction affecting some near surface deposits, especially on sloping ground (Hughes et al, 1998).

Database systems

Databases are a collection of related data. They are actually nothing more than computerised record-keeping systems or, in other words, a type of electronic filing cabinet. The initial reason for the development of databases was to centralise all available data in a common format. This immediately simplifies any comparative studies undertaken, assists in the identification of any problems with existing data, and highlights gaps in the data. The benefit of this system is that the users can perform a variety of operations rapidly.

One of the fundamental features of a database is that it allows data to be shared between different applications. All access to the data is performed through a DataBase Management System (DBMS) which organises and structures the data in such a way that they can be retrieved and manipulated by users and application programs. A DBMS can be simply defined as a program or software that allows the user to define, create and maintain a database and provides controlled access to this database (Elmasri and Navathe, 2000). Databases are constructed from data models, which provide database users and the DBMS with a way to access and structure the data. A data model is a collection of the concepts that can be used to describe a set of data, the operations to manipulate the data, and a set of integrity rules for the data. Hence data models can be used to categorise a DBMS. Various data models have been developed and used over the years and are briefly reviewed below (Bowers, 1993; Connoly and Begg, 1999):

- Hierarchical Data Model. In this model the data are presented as records which are linked together like a family tree (Figure 2). The records are organised in child/parent relationships which means that each part is linked to its subpart. The hierarchical model uses the structure of a hierarchy, where a data type can have several "children" data types, but each child can have only one "parent" data type. The retrieval of the data involves navigation through the hierarchical structure from the "parent" to the "child" in a pre-order

traversal. This model is not suitable for data with complex structures as its structure is not flexible to changing data and access requirements.

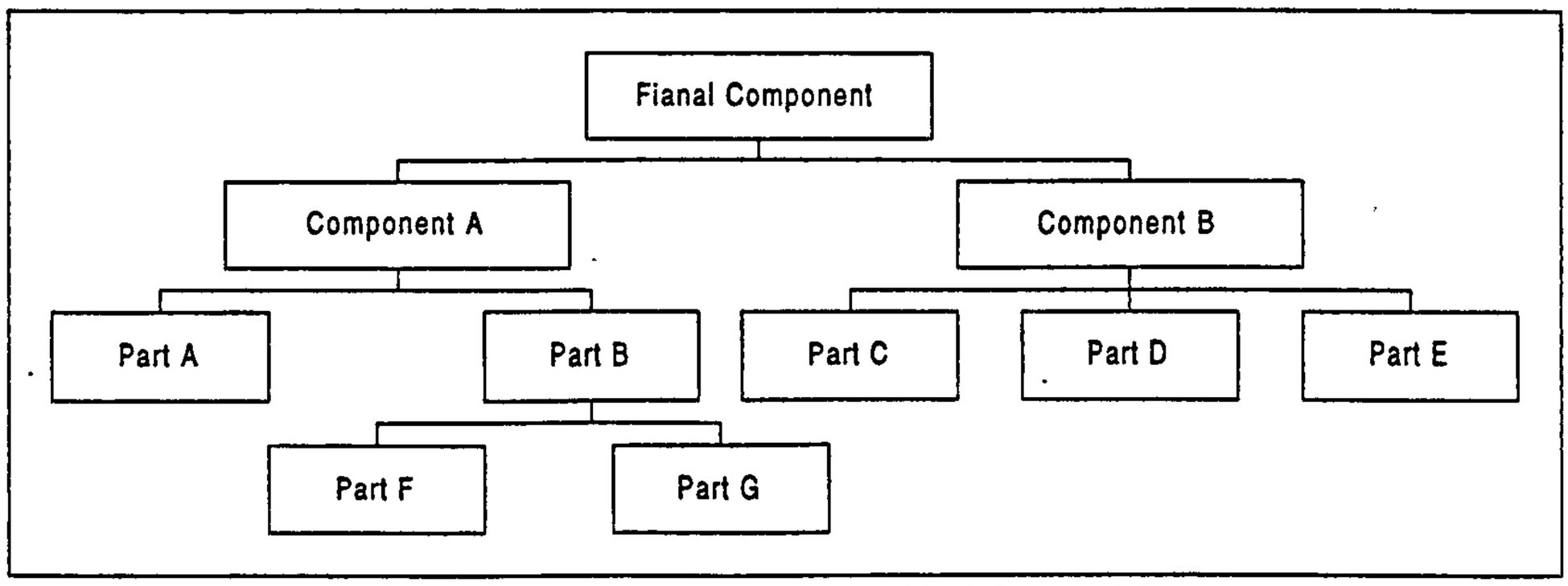

Figure 2: Example of a Hierarchical data model

- Network Data Model. This model is an extension of the Hierarchial Data Model. The Network Data Model allows data records to participate in multiple child and parent relationships known as sets (Figure 3). Explicit links or storage addresses, called pointers, contain the location of a related record and must be maintained at all times. To use the Network Model, the user is required to be familiar with the structure of the database and to know where the data are stored. The multiple parent/child relationships allow a network database to represent data with complex structures. The disadvantage of this model is that the relationships and the structure of the records are pre-set and therefore not very flexible. The user can only access the data from a given point in the network.

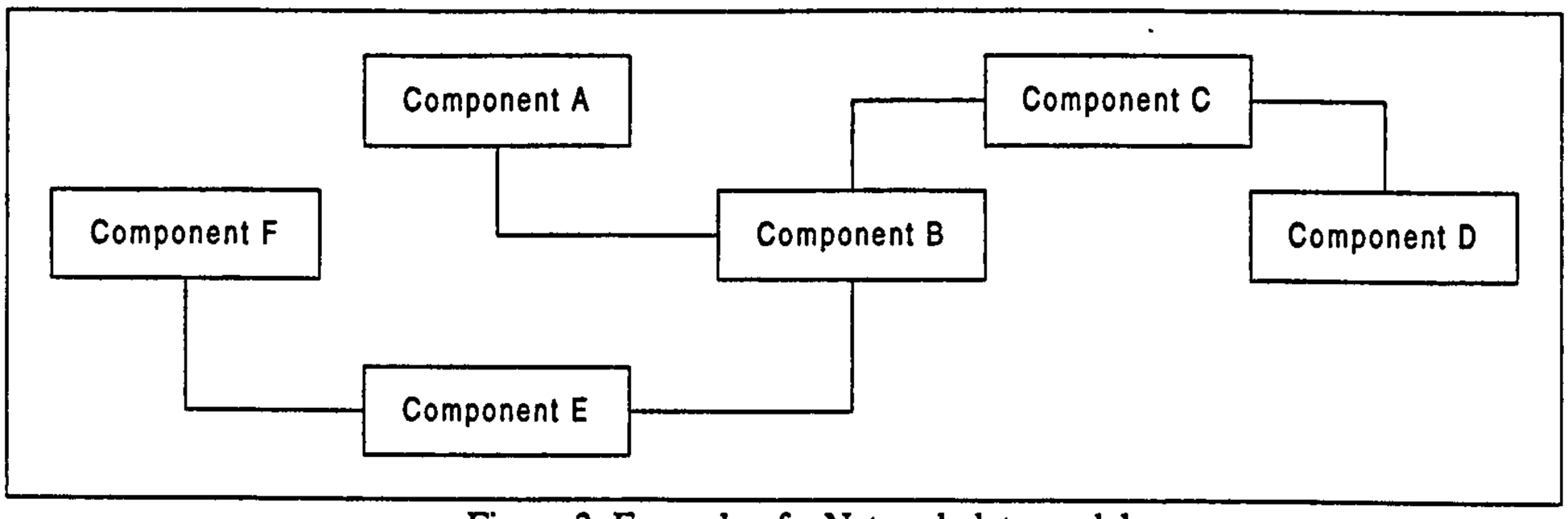

Figure 3: Example of a Network data model

- Relational Data Model. A relational database model is a collection of related data and is based on the concept that data are stored in two-dimensional tables, called relations (Figure

4). Each row in the table represents a record and each column represents a field. The entire table is roughly equivalent to a file. Common data columns known as primary key fields relate the tables to each other. Primary key fields are fields that uniquely identify each record, such as those underlined in Figure 4. It can be said that a relational database is a database where all data are strictly organised as tables of data values, and where all database operations work on these tables. A relational database does not need structures like a hierarchical or network database, but can represent parent/child relationships which are represented by the data values within the database tables. This model is flexible for changing data and access requirements.

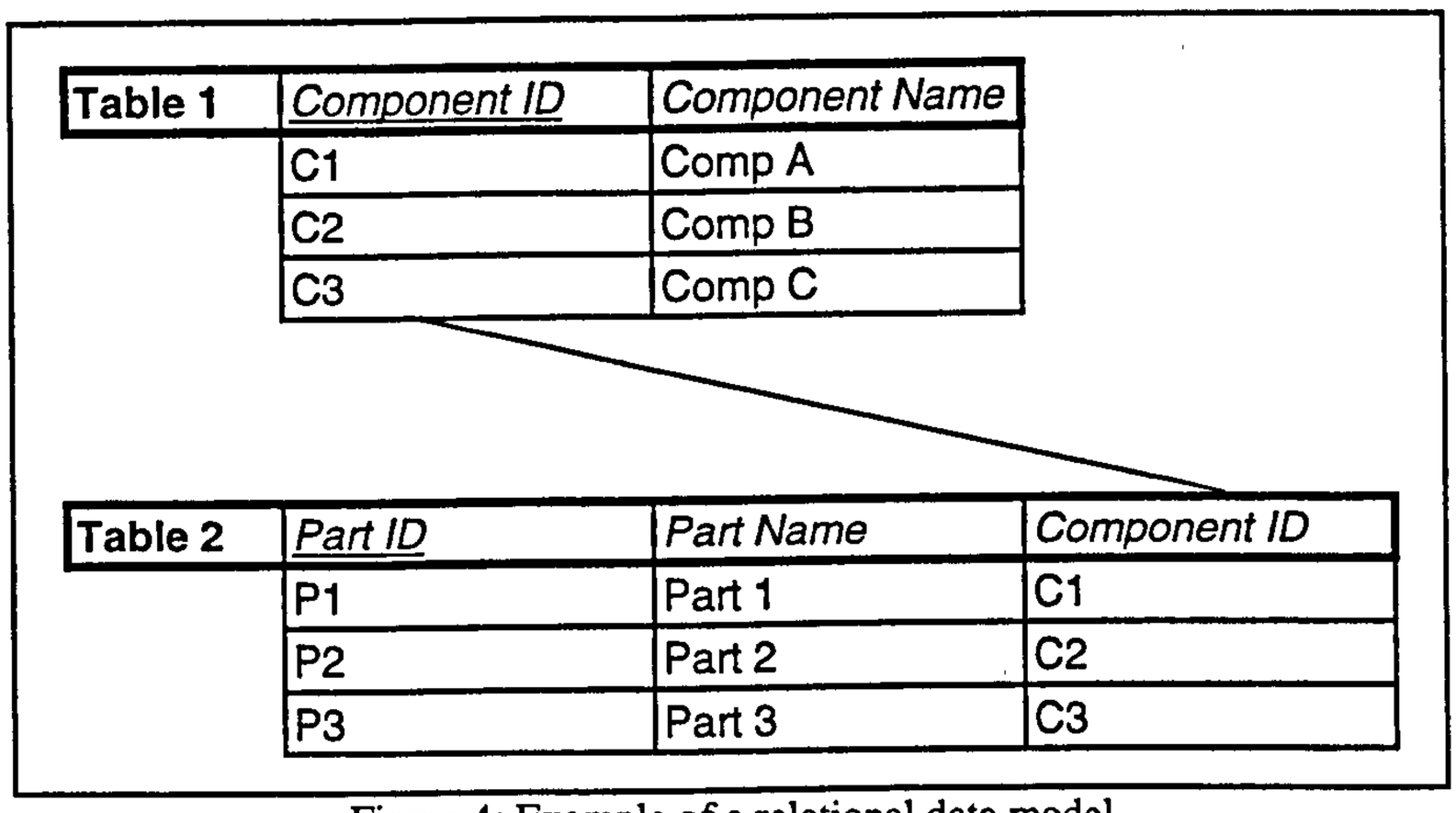

Figure 4: Example of a relational data model

The relational model allows a high degree of data independence and provides substantial grounds for dealing with consistency and redundancy problems, and was therefore found to be most suitable for developing NETDATA.

The relational model depends on the type and the condition of the data. The three main relational types are:

- A one-to-one relationship is used when a record from one table is related to no more than one record from another table.
- A one-to-many relationship is used when a record from one table is related to many records from another table.

- A many-to-many relationship is used when a record from one table is related to many records from the second table, and many records from the second table are related to many records from the first table.

The software

A good database management system should not only be able to store any amount of data but also should provide entry screens to get data in, search screens to find data, and output screens to present the data found. It should also be able to process data to enhance the system (Mallender, 1995). It is important that the management system of a database should have a flexible query system to allow people to view the database and retrieve information in various ways.

In order to meet these needs, it was important to choose a DBMS that provides flexibility and, more importantly, to effectively represent the relationships between the various datasets given the varying quality of the data. Several commercial database packages have been especially designed for the storage of geotechnical parameters, for example gINT (gINT, 2002), HoleBASE II (HoleBASE II, 2002) and SID (SID, 2002). These packages provide standard options of a relational database system for storing data and performing tasks such as searching for data, preparing reports, or providing graphical outputs like borehole logs, but their use is restricted to the fixed structure of the software.

Other programs and software are also available commercially which can be used as a platform for the design of databases and data representation. Unlike the purpose written geotechnical database packages, these software platforms make it possible to design the structure of a database according to the needs of a project, and have the advantage of being more flexible for applying structural changes to the design of a database if required. For these reasons it was decided to design a database specifically for this study with a user-friendly software that is widely available.

One popular software is Microsoft Access, which is widely available and has a user-friendly interface. More importantly the interface is common to a suite of packages within Microsoft Office which is familiar to many engineers. This package includes Excel, which allows further manipulation of data. The Microsoft Access interface contains a list of menu options

which eliminates the need for memorising specific commands and syntaxes for performing various tasks. Its interface is also form based which means that forms can be designed and displayed for data entry and editing. The graphical interface of this software enables the user of the database to specify a query by manipulating available diagrams. In addition to the above, Microsoft Access provides options such as shortcut keys which users may define for performing tasks such as printing, the possibility of creating authorised user accounts for multiple users and setting passwords for the security of databases. Microsoft Access provides a stable and consistent platform which is required for the effective modelling of the complex relationships between diverse datasets.

Microsoft Access allows interactive definition of relationships between tables which can specify referential integrity constraints via the relationship window. This allows design modifications to be made without affecting the integrity of the data. Another useful tool available in this software is that it can keep up with the fast advancing technology. Special options are provided by the software which can convert the old version of the database into newer versions, for instance from Microsoft Access 97 to Microsoft Access 2000. This option not only keeps the format of the data as they are, but can also transfer all existing objects without changing the structure of the database into the new format. This is a key requirement given that the database will expand over the years as ground investigations continue. The above advantages of Microsoft Access led to the conclusion that it was the best solution for the design of NETDATA to create a Relational DataBase Management System (RDBMS).

The structure of NETDATA

One of the key ideas of a database is that it should represent information by modelling reality. This means that a database should be a picture of a part of reality, which could be updated as new facts are discovered (Rasmussen, 1995). To maintain efficiency and reliability it is important that the data follow a standard format. It is fundamental to ensure that the data in a database are held in a coherent way to allow all the potential links and queries to be made correctly and efficiently. The best way to achieve this is to build a data model (also called a logical model).

Data modelling is used to produce an accurate representation of the information needs of the system. It allows the designer to describe the data in a system and explain it to others. There

are two basic types of model. First, a logical model that views a system independently of specific hardware or software; this permits objective decisions to be made. This is effectively a map of the data, and their elements and relationships. Second, a physical model, is developed for implementation within the chosen software and represents the logical model adapted to the constraints of specific hardware and software. The data model records the two main characteristics of the data within the system which are the fields used in the tables and the relationship between the tables. Fields within tables identify items stored in the database. They can be thought of as something real within the system, such as a borehole or a sample taken from that borehole. The purpose of identifying the fields is to define the types of objects the system must deal with, and which individual items of data are associated with each type of object. The fields are put into tables, which will relate to each other in various ways. The relationships shown in a data model record how data are interconnected.

One logical model which has been successfully introduced and adopted in most UK specifications for ground investigations is the Association of Geotechnical and Geoenvironmental Specialists (AGS) model (AGS, 1999). Files used in this model should contain basic data normally found in a factual report. These include exploratory hole records, and the test data, all of which should be consistent with the relevant British Standards and other recognised documents. The file format is intended to provide a wide level of acceptance and, in view of this, it is considered that the data should be transmissible using American Standard Code for Information Interchange (ASCII) files.

Data Groups have been chosen to relate to specific elements of data, such as project information and exploratory hole details. Fields within each Data Group identify items such as test details or test results. Two types of Data Fields defined by the AGS format are the KEY Fields and the COMMON Fields. The KEY Fields must be included in every Data Group within a file. They are important for maintaining data integrity. Data entered into KEY Fields must be unique in each GROUP and the corresponding entries must be made in the PARENT GROUP. All other fields within a Group are called COMMON Fields.

The AGS Format Data Groups are organised in a hierarchy with an inverted tree like structure as shown in Figure 5. At the top of the tree is the HOLE Group, which contains information about all the boreholes, and all other Groups lie below this. One of the Groups immediately

below HOLE is SAMP, which contains details of all samples, all the laboratory testing Groups lie below SAMP. The HOLE Group is termed the "parent" Group of SAMP. The PROJ Group, which contains details of all projects, sits above the tree, and has a general purpose. It must always be included in an AGS Format submission as it defines the project.

The structure of NETDATA follows this model, and the tables and fields have been designed following the rules and standards of the AGS Format. The fields in each table are logically related and a unique field in each table is chosen as the Primary Key of the table which is used to join one table to other tables. Reference fields such as PROJ_REF, HOLE_REF and SAMP_REF are examples of KEY Fields that are used to relate tables in NETDATA. Following the AGS model, the relationships between the tables of the database, represent a parent to child relationship. The structure of NETDATA and the connections between the tables in the relationship window is shown in Figure 5. The TBL-PROJ, which contains the fields that define the project, is the first table. Each project can have many boreholes, therefore a one-to-many relationship (1 - ∞) is created between TBL-PROJ and TBL-HOLE. Several samples can be taken from each borehole therefore the tables TBL-HOLE and TBL-SAMP are connected to each other with a one-to-many relationship. Each sample may be used for different laboratory tests (more than one test) and therefore a one-to-many relationship has been created between TBL-SAMP and the test result tables. The description of the fields used in various tables of NETDATA as shown in Figure 5 can be found in the Appendix of this paper. These are based on a Data Dictionary introduced in the AGS Format, which is applied in NETDATA.

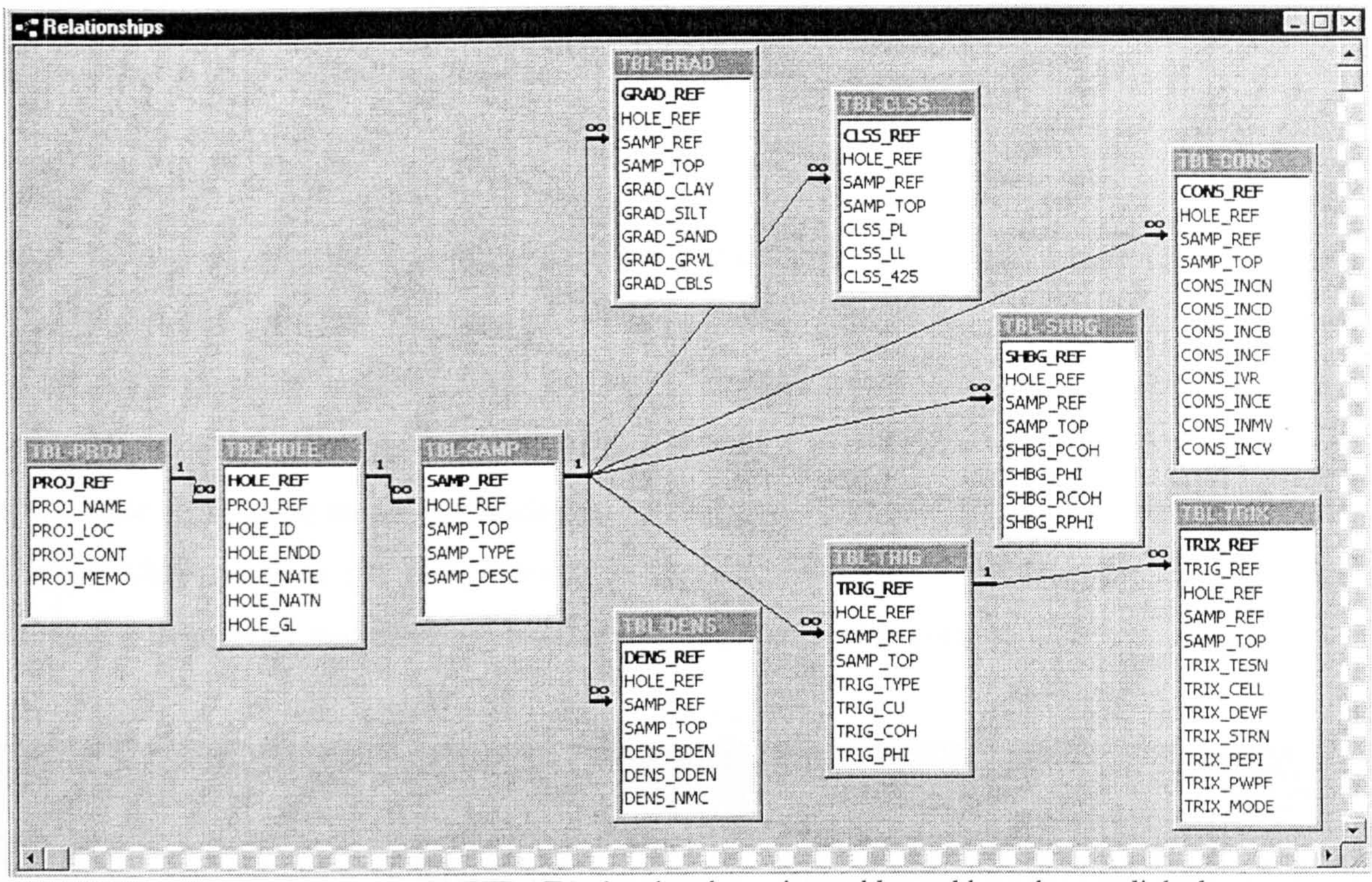

Figure 5: The structure of NETDATA showing the various tables and how they are linked (see Appendix for full descriptions of fields)

To allow the file formats to be more easily recognised by the non-specialist, a Data Dictionary has been prepared that defines the Groups, Fields and Units used in the database. Descriptions of the fields and units used for various parameters are automatically displayed in all objects of NETDATA. A dictionary can be a powerful tool for assisting database validation. It can help to eliminate erroneous values being entered in the database, or unauthorised and undocumented codes being created by users. The user of a database should be confident that the data held are reliable. Dictionaries ensure that only valid values are used within the database. A dictionary also helps the user to understand clearly what is meant by a given term or set of terms used within the database (Giles et al, 1997).

Features of NETDATA

A good database system should provide options for entering, editing, searching for, displaying and presenting data. NETDATA not only stores and manages data efficiently and in a controlled manner, but is also an extremely powerful tool for manipulating and presenting

data. It stores data in a rigidly defined way, but allows multiple views of the data, as individual records or in virtually limitless permutations. Objects such as tables, forms, queries and reports are included in the database for these purposes and are described below:

Tables

Tables (Figure 6) store the data of the database and organise them into columns (called fields) and rows (called records). They are the most important part of the database. The tables and the fields within the tables follow the AGS Format which has been described earlier. Microsoft Access makes it possible to set properties for each field within the table which can be used for formatting data. This includes defining the field format and size, setting default values, validation rules for data entry (e.g. entering no more or no less than 6 digits for northing and easting), and allowing/disallowing duplicate value entries.

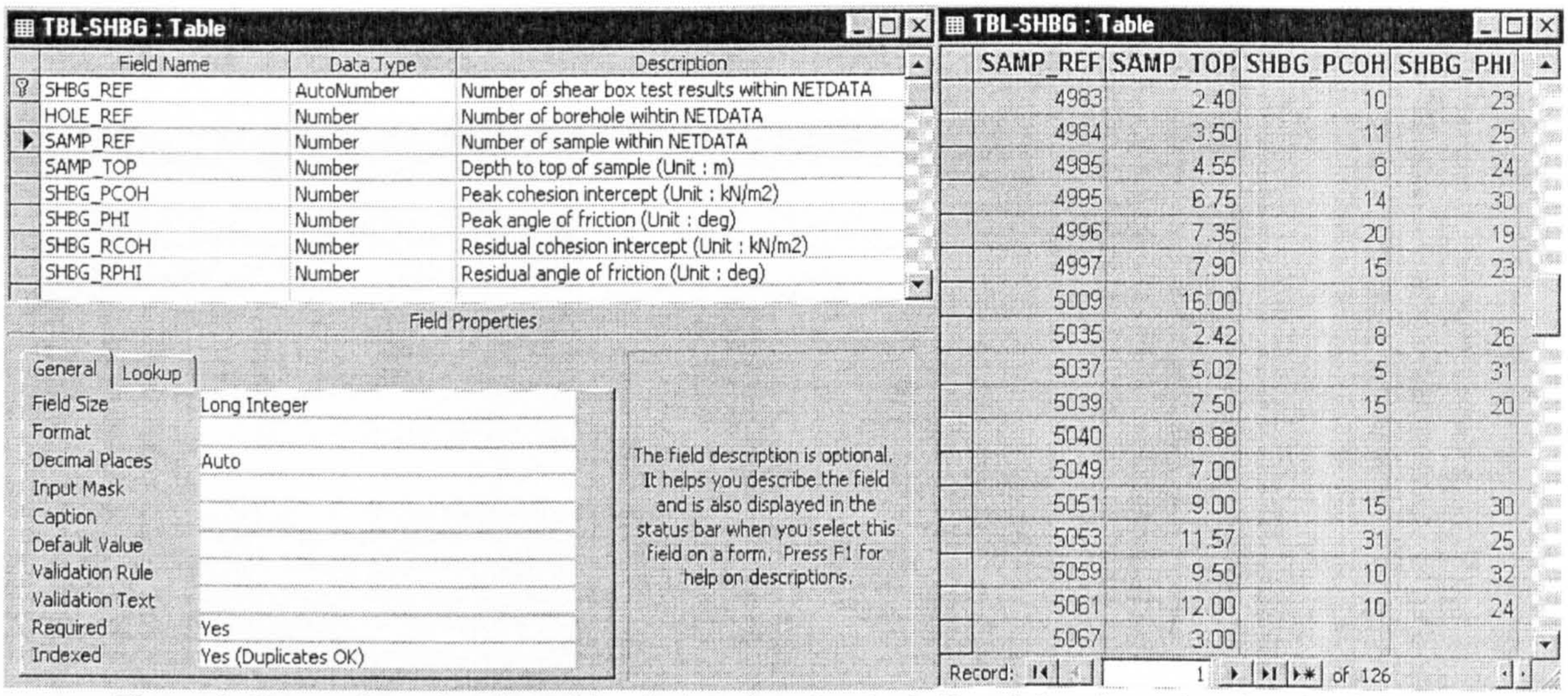

Figure 6: Screen shots showing tables design view (on the left) and datasheet view (on the right)

Forms

It is possible to view, enter or edit data directly from relevant tables of the database, but several forms have also been designed to make the process of viewing, entering or editing data easy for the user. The procedure for the above action is explained briefly on the forms as shown in Figure 7. Users are prompted with information messages for requirements and data formats, or with error messages when wrong formats have been used for data entry. Some of the data fields are filled in automatically by the DBMS which retrieves matching data from the database. This will provide additional help for the user and prevent unwanted mistakes. For instance, if new test results are to be entered into the database via forms, the user only needs to input the sample reference into the provided fields. The system will then automatically produce a reference number for the test result and fills in the relevant borehole name, project and location information. The remaining fields must be filled in by the user and the entries saved.

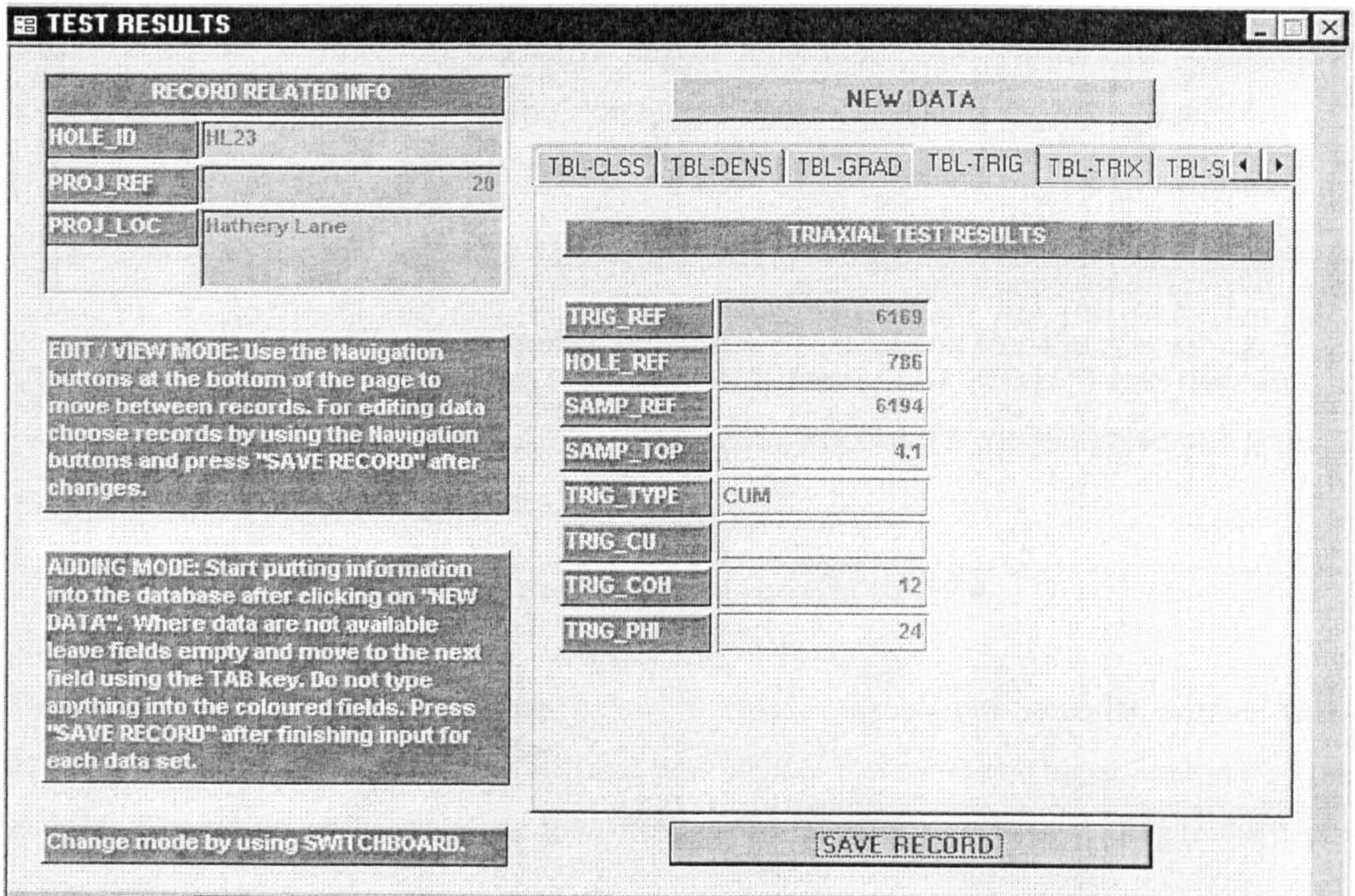

Figure 7: Screen shot showing a form used to view, edit or add data in the database

Queries

The main use of a database is to gain access to the stored data to find the required information. Queries make it possible to ask for specific information about the required data. There are two main reasons for wanting to retrieve data from a database. Firstly to view or print some or all of the data, and secondly to analyse data using different techniques, in which case the data can be exported into a data processing system such as Excel.

The ability to query the data in a variety of sophisticated ways is one of the extremely powerful features of a RDBMS. The data held within the RDBMS can be interrogated by using Structured Query Language (SQL), which is an industry standard non-procedural language used to manipulate and control databases. The language uses a series of SQL statements to perform a variety of tasks such as querying data and controlling access to data. This option is appropriate for more advanced users. The code can be used to design complex queries by joining tables (using the JOIN command), select data fields from tables that hold the required data (using the SELECT and FROM commands), define conditions for the data search (using the WHERE command), or impose a preferred order on the data according to specific criteria (using the ORDER BY command). A sample code is shown in Figure 8.

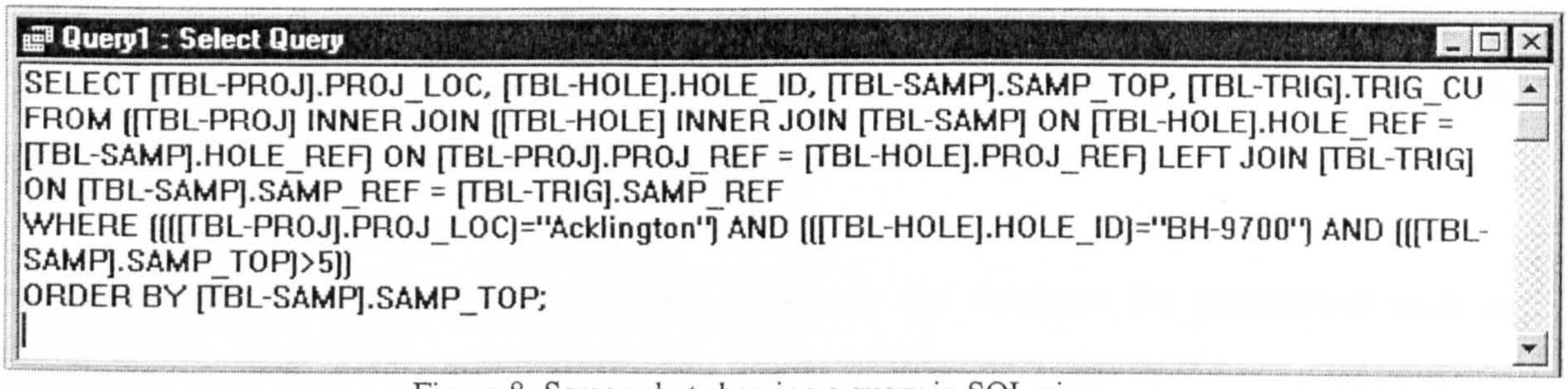

Figure 8: Screen shot showing a query in SQL view

The above code searches for samples below "5 metres" taken from borehole number "BH-9700" from a location in "Acklington" and will display the undrained shear strength of the samples. The results of the query are then displayed in ascending order according to the depth of the sample.

A less daunting way of interrogating the database is the use of the graphical interface of the Microsoft Access software which makes it possible to ask questions about the data stored in the tables of the database without the need for writing SQL codes. This interface allows users to view and choose from the list of tables and fields, and to set criteria to find certain data. Figure 9 shows the graphical version of the SQL code shown in Figure 8.

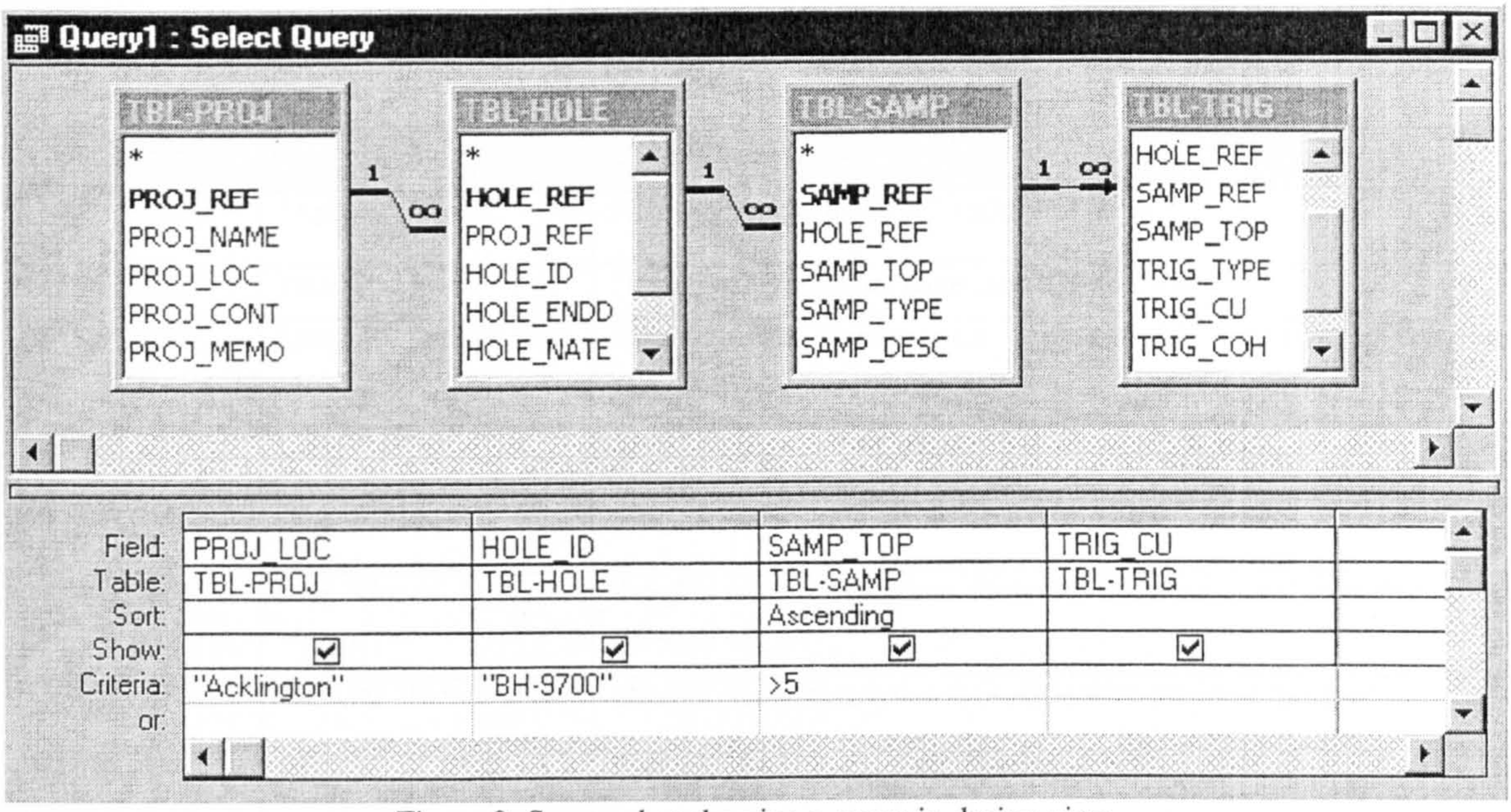

Figure 9: Screen shot showing a query in design view.

Several queries have been designed and prepared to retrieve data from the tables within NETDATA for analysis or printing which include the following:

- QRY-INDEX: this query can be used to search the database for parameters such as plasticity, particle size and density. The results of this query are used to produce reports on the index properties, and classification graphs.
- QRY-CONS: this query will search the database for the results of a consolidation test and to prepare consolidation test reports.
- QRY-SHEAR: this query can be used to search for shear strength parameters derived from either triaxial tests or shear box tests. The results are also used to prepare strength test reports.

Using these queries the user can search for parameters related to samples from certain locations or projects by setting the required criteria as shown in Figure 5. The user of the database is also able to easily design new queries, if required, using available "wizards" within the Microsoft Access software. An expression, or a combination of expressions, may be used to search for specific data within the database. Table 1 shows some expressions that could be used in queries:

Table 1: Some expressions used in queries

Field	***Expression***	***Displays***
PROJ_LOC	"Acklington"	Projects from the location "Acklington" only.
PROJ_LOC	Like "B*"	Projects whose location name start with the letter "B"
PROJ_LOC	Like "A*" Or Like "B*"	Projects whose location name start with "A" or "B". (Includes both alternatives)
SAMP_DESC	Like "*gravel"	Samples whose description end with the word "gravel".
SAMP_DESC	Like "*SAND*"	Samples whose descriptions include the word "SAND".
SAMP_DESC	Like "Grey*" And Like "*CLAY*"	Samples whose descriptions start with the word "Grey" and also include the word "CLAY".
SAMP_DESC	Not Like "*boulder*"	Samples whose description does not include the word "boulder"
CLSS_PL	>15	Samples whose plastic limit is greater than 15.
CLSS_PL	=<20	Samples whose Plastic Limit is equal or smaller than 20.
GRAD_CLAY	<>10	Samples whose clay content is not equal to 10.
GRAD_SILT	=>15 And <20	Samples whose silt content is equal or bigger than 15 and smaller than 20
GRAD_SAND	Between 15 And 20	Samples whose sand content is between 15 and 20
GRAD_GRVL	Not 25	Samples whose gravel content is not 25
GRAD_CBLS	Is Not Null	Samples whose cobble content is not null

To make it even easier to search for data, a special form has been designed for filtering data within NETDATA (Figure 10). The user of the database can then set the criteria for various parameters without using Microsoft Access queries.

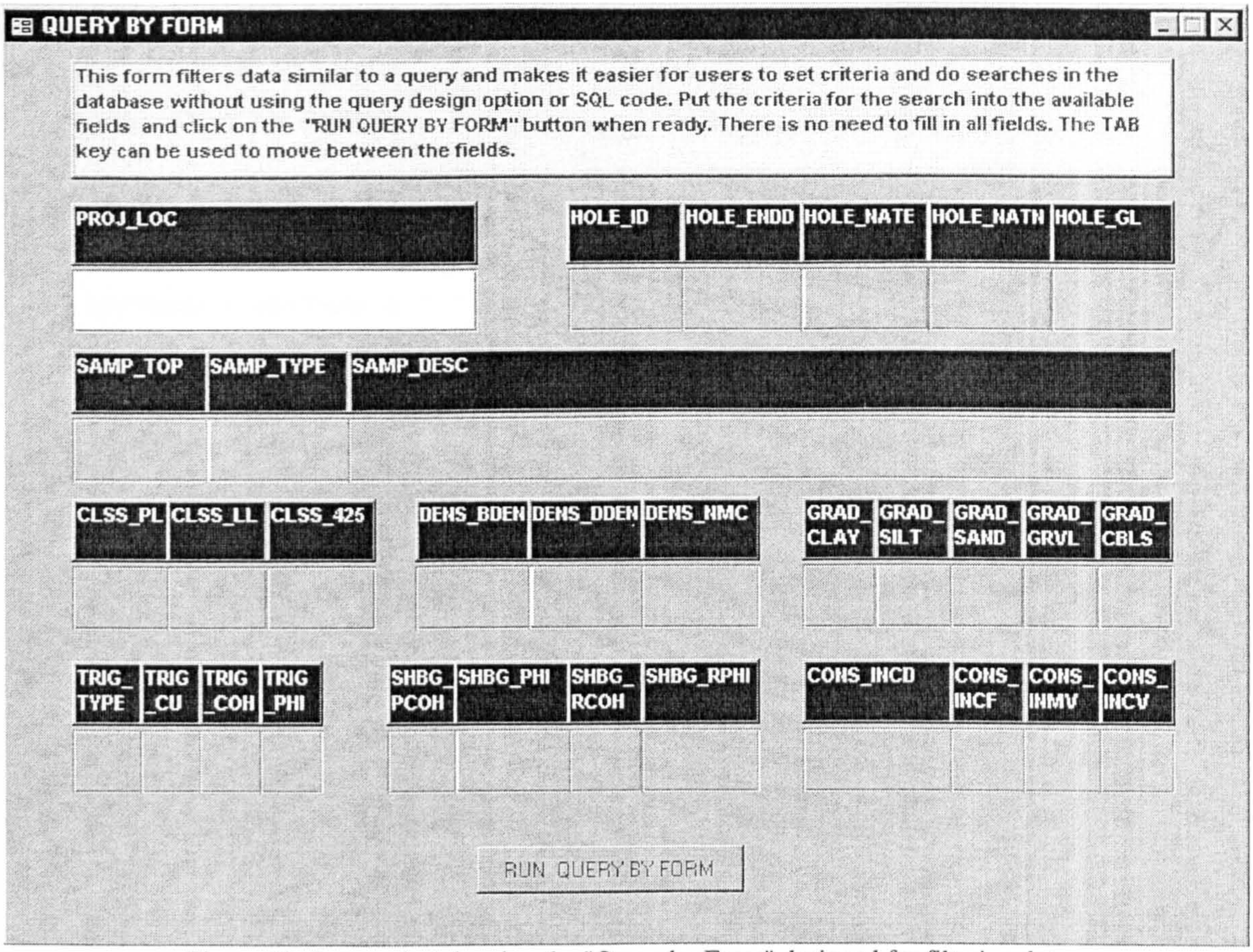

Figure 10: Screen shot showing the "Query by Form" designed for filtering data

This specially designed form combines Visual Basic and SQL commands together (see Figure 11), to create a powerful and easy to use tool for searching and querying of data. Boxes are provided within the form representing fields from various tables such as the location, borehole name, sample depth and description, and test results. The user only needs to put the required criteria shown in Table 1 into the boxes and the combined code will filter the data from the tables automatically. This makes the search for data easier for anyone not familiar with the use of queries or their design methods.

```
Private Function BuildSQLString(strFieldName As String, varFieldValue As
Variant, intFieldType As Integer)
   Dim strTemp As String

   strTemp = "[" & strFieldName & "]"
   If isOperator(varFieldValue) Then
     strTemp = strTemp & " " & varFieldValue
   Else
    · Select Case intFieldType
         Case dbBoolean
           strTemp = strTemp & " = " & CInt(varFieldValue)
         Case dbText, dbMemo
           strTemp = strTemp & " LIKE " & QUOTE & varFieldValue & "*" &
QUOTE
         Case dbByte, dbInteger, dbLong, dbCurrency, dbSingle, dbDouble
           strTemp = strTemp & " = " & varFieldValue
         Case dbDate
           strTemp = strTemp & " = " & "#" & varFieldValue & "#"
         Case Else
           strTemp = ""
      End Select
   End If
   BuildSQLString = strTemp
End Function
```

Figure 11: Sample code combining Visual Basic and SQL commands (adapted from Getz et al, 1994)

Results of queries can be displayed in forms, delivered as new database tables, printed as reports, or saved in other formats such as ASCII files or HTML files. Data can also be exported into other software such as Microsoft Word and Microsoft Excel.

Reports

Reports are another feature of the database, which have been prepared within NETDATA for summarising and presenting data from tables and queries. They may be printed on paper or displayed on the computer screen. The report facility in the software can be used to organise and present data in groups, carry out calculations, produce graphs and put the data in required formats. Several reports have been prepared based on query results and are available in NETDATA. These produce summary sheets of project descriptions, sample descriptions and various test results. An example of a report is shown in Figure 12.

SAMP_REF	HOLE_ID	SAMP_TOP	SAMP_TYPE	SAMP_DESC	CLSS_PI	CLSS_LL	CLSS_PI	CLSS_425	DENS_NMC	DENS_BDEN	DENS_DDEN
4796	BH-9700	1.50		Firm reddish brown mottled light grey weathered occasionally laminated fine sandy very silty CLAY with occasional fine gravel and fine organic fibers (rootlets).	28	58	30	100	22.1	1.94	1.59
4797	BH-9700	2.50		Stiff becoming very stiff with depth, dark reddish brown slightly laminated slightly sandy CLAY with some sandstone, limestone and occasional mudstone fragments	20	37	17	88	16.8	2.13	1.83
4798	BH-9700	3.51		Stiff becoming very stiff with depth, dark reddish brown slightly laminated slightly sandy CLAY with some sandstone, limestone and occasional mudstone fragments	18	36	18	78	18.4	2.11	1.78
4799	BH-9700	4.60		Stiff becoming very stiff with depth, dark reddish brown slightly laminated slightly sandy CLAY with some sandstone, limestone and occasional mudstone fragments	19	37	18	81	16.9	2.13	1.82

Figure 12: Screen shot showing part of a sample report produced in NETDATA

The first page of each report contains a list of the fields used in the report with the relevant description and units as shown in the Appendix of this paper. New reports can be designed based on data from tables or results of queries by using available "wizards" within the Microsoft Access software.

Other objects

Other forms are also available to process the data within the database. Extensive use has been made of the capabilities of Microsoft Access in communicating with other software. Available options within Microsoft Access make it also possible to transfer data from tables and queries into Microsoft Word documents or Microsoft Excel spreadsheets. A Microsoft PowerPoint presentation is also prepared and can be run from inside the database, which includes

background information about the geology of Northern England. A list of all the above objects and their functionalities can be found in Table 2.

Table 2: Showing list of objects within NETDAT used for data processing and presentation.

Object Name	*Functionality*
FRM-CHRT_INDX	This form is used to plot the plasticity chart using Microsoft Excel and based on the results of QRY-INDEX query. It also calculates parameters such as the Plastic Index and the Liquid Index from the query results. These parameters are not included in any table.
FRM-CHRT_PSD	This form can be used to plot the grading classification triangle using Microsoft Excel and based on the results of QRY-INDEX query.
FRM-CHRT_OTHR	A form designed as a flexible tool, which enables the user of the database to transfer the results of QRY-INDEX to an Microsoft Excel sheet and plot any required chart.
INTRO.PPS	A Microsoft PowerPoint presentation which includes background information about the geology of Northern Englandand and pictures of various opencast sites focusing on weathered and unweathered till layers on face exposures.

Help Facility

An essential part of the design procedure is to provide help for carrying out various tasks within the database. Using Microsoft Help Workshop Version 4 an online help facility was designed for NETDATA in order to eliminate the need for a physical manual. This hypertext help system along with the more technical help options provided by the Microsoft Access software related to database design, provides a useful facility for users who are not familiar with this database. The NETDATA help facility contains information about all objects used in the design of the database and provides help options to carry out tasks such as running available queries, adding or editing data in different ways, methods of using the specially designed forms, exporting data, producing reports etc. Following the AGS Format, codes of abbreviations are used in a number tables in order to ensure consistency in terminology and for brevity. These are also described in the help facility of the database. The user can simply choose from the available help options or search for help on a specific topic (see Figure 13).

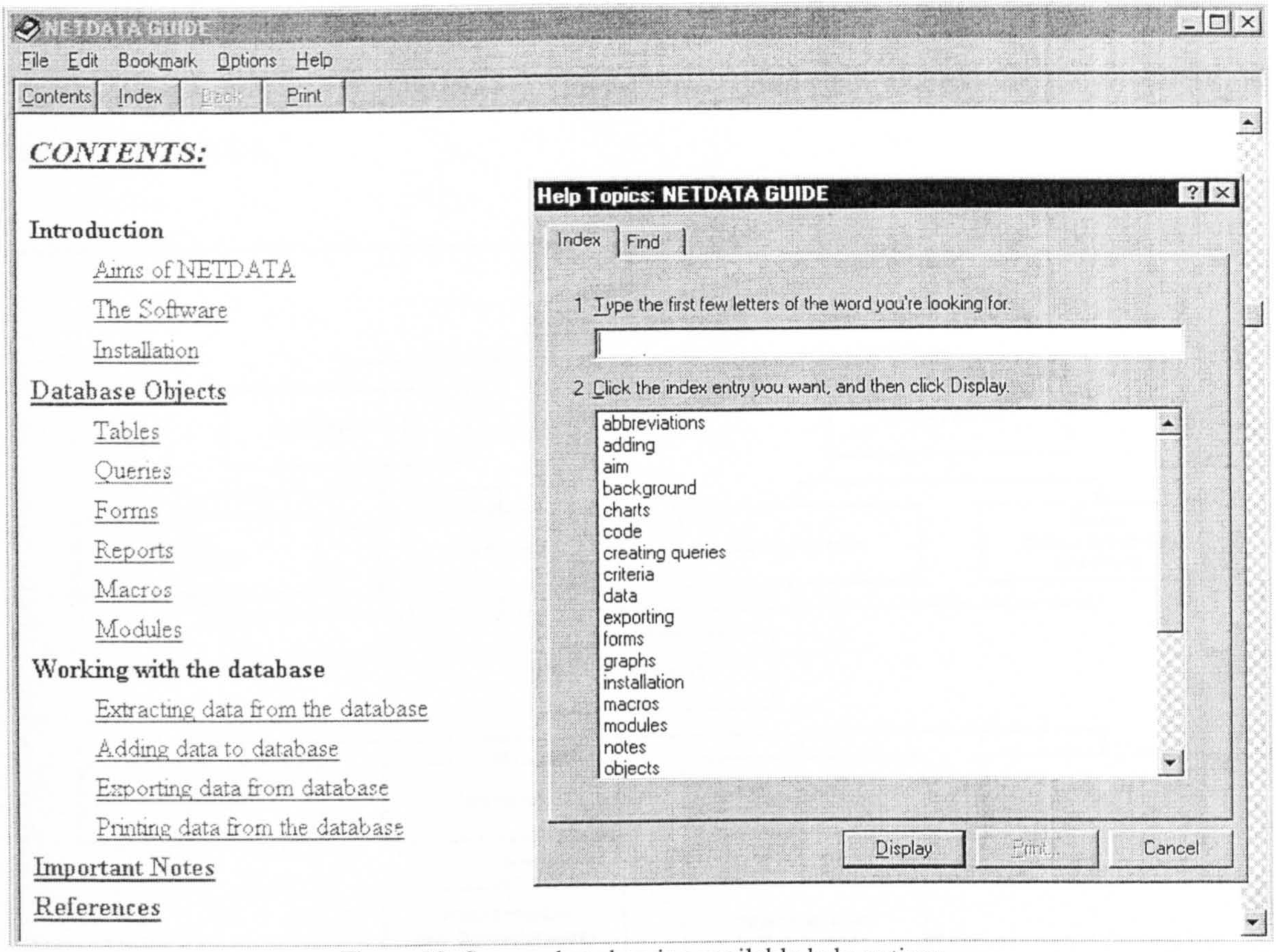

Figure 13: Screen shot showing available help options

Applications of NETDATA

As stated previously one of the main aims of the collection of site investigation data is to construct a three dimensional image of the superficial deposits in relation to engineering projects. Data can be retrieved for a specific vertical or horizontal location. The data stored in the database can be exported to other software in order to build a graphical model of the local stratigraphy. Various tools and options in NETDATA make the extraction of data from the database very easy and effective. The data can be used for analysis with different techniques. The presence of data from various locations makes a comparative study of the database possible. Robertson et al (1994) published results of their evaluation of the classification and strength data for Northumberland tills based on a simple spreadsheet analysis exercise.

NETDATA is currently being used to carry out a more in-depth analysis for the glacial deposits for the whole of Northern England, and the results of this will be published later.

The flowchart shown in Figure 14 summarises the various steps and available options for the user of NETDATA.

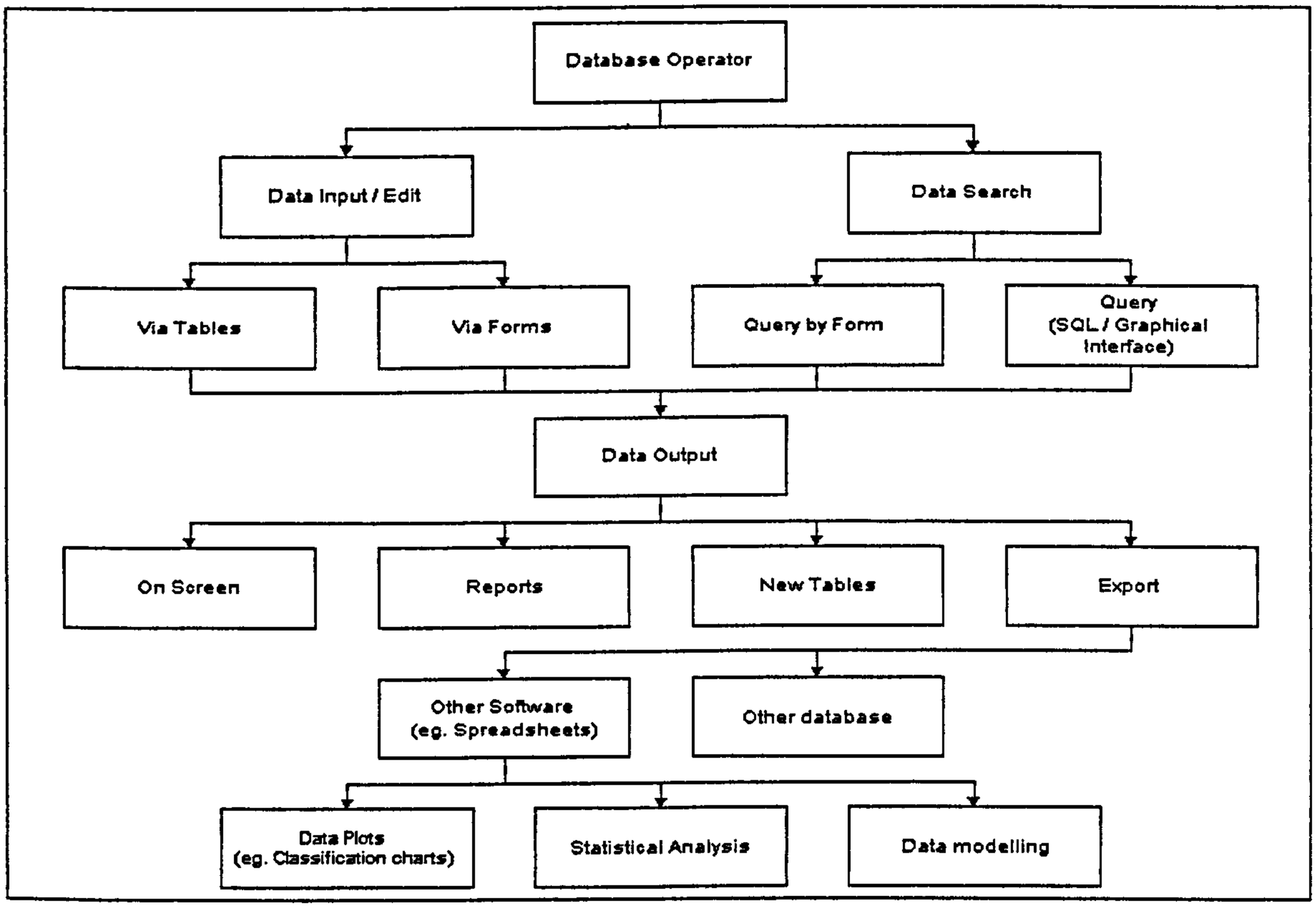

Figure 14: NETDATA operation flowchart

Conclusion

NETDATA is a modern computer-based system which is used to store and manage a large quantity of geotechnical information. The volume and detail of available data suggested that management within a traditional paper system was likely to be less efficient than a computer-based approach. Paper products are less flexible and more time-consuming where it is required, for example, to separate chosen elements or change scale, to vary the area covered or to integrate other information. It is difficult to update conventional paper data holdings when new evidence or improved interpretations become available. These requirements make a

digital system such as NETDATA essential. However, an archive of paper records in support of the database and to satisfy their Quality Assurance requirements is maintained.

Microsoft Access version 97 was found to be a suitable RDBMS with an appropriate interface and software tools for the design of NETDATA. The AGS Format is a data model which is widely used in most specifications for ground investigations. This model was followed in the design of the database in order to put the available data into a standard format and ensure consistency and coherency between the data. Extensive use was made of the software capabilities and tools to provide a user-friendly and secure interface for handling data, and for carrying out various tasks such as searching for certain data and presenting them in different formats.

NETDATA is a dynamic and evolving system, and continues to expand both in terms of the variety and the volume of the data that it holds. It stores and centralises available data for the glacial tills of Northern England in a standard format which is easy to update. New datasets will continue to become available and be incorporated into the database. This makes data from various locations accessible for comparison, and should lead to an extensive analysis of the parameters, which are used to define the geotechnical properties of the North of England glacial tills. The results could then be used to re-analyse some earthwork failures, such as excavated slopes and spoil mounds, where records are available. This should lead to a better understanding of the engineering behaviour of glacial tills, and more suitable parameter selection for engineering design.

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Appendix: Description of the fields within the tables of NETDATA

Field name	*Data type*	*Description*
PROJ_REF	Auto-number	Number of project within NETDATA
PROJ_NAME	Text	Title of project
PROJ_LOC	Text	Location of site
PROJ_CONT	Text	Name of contractor
PROJ_MEMO	Text	General project remarks or comments
HOLE_REF	Auto-number	Number of borehole within NETDATA
HOLE_ID	Text	Exploratory hole name / number
HOLE_ENDD	Date / time	Hole end date
HOLE_NATE	Number	National grid easting of hole
HOLE_NATN	Number	National grid northing of hole
HOLE_GL	Number	Ground level relative to ordnance datum (m)
SAMP_REF	Auto number	Number of sample within NETDATA
SAMP_TOP	Number	Depth to top of sample (m BGL)
SAMP_TYPE	Text	Sample type
SAMP_DESC	Text	Sample description
CLSS_REF	Auto number	Number of index text results within NETDATA
CLSS_PL	Number	Plastic limit (%)
CLSS_LL	Number	Liquid limit (%)
CLSS_425	Number	Percentage passing 425 (micro m) sieve (%)
GRAD_REF	Auto number	Number of grading test results within NETDATA
GRAD_CLAY	Number	Clay fraction (<0.002 mm) - (%)
GRAD_SILT	Number	Silt fraction (>0.002 , <0.06 mm) - (%)
GRAD_SAND	Number	Sand fraction (>0.06 , <2 mm) - (%)
GRAD_GRVL	Number	Gravel fraction (>2 , <60 mm) - (%)
GRAD_CBLS	Number	Cobbles and boulders fraction (>60 mm) - (%)
DENS_REF	Auto number	Number of density test results within NETDATA
DENS_BDEN	Number	Bulk density (Mg/m3)
DENS_DDEN	Number	Dry density (Mg/m3)

DENS_NMC	Number	Natural moisture content (%)
TRIG_REF	Autonumber	Number of triaxial test results within NETDATA
TRIG_TYPE	Text	Triaxial test type
TRIG_CU	Number	Value of undrained shear strength (kN/m2)
TRIG_COH	Number	Cohesion intercept associated with TRIG_PHI (kN/m2)
TRIG_PHI	Number	Angle of friction for effective shear strength triaxial test (deg)
TRIX_REF	Autonumber	Number of triaxial test detail within NETDATA
TRIX_TESN	Number	Triaxial test / stage number
TRIX_CELL	Number	Total cell pressure (Unit : kN/m2)
TRIX_DEVF	Number	Deviator stress at failure (kN/m2)
TRIX_STRN	Number	Strain at failure (%)
TRIX_PEPI	Number	Porewater pressure at start of shear stage (kN/m2)
TRIX_PWPF	Number	Porewater pressure at failure (kN/m2)
TRIX_MODE	Text	Mode of failure
SHBG_REF	Autonumber	Number of shear box test results within NETDATA
SHBG_PCOH	Number	Peak cohesion intercept (kN/m2)
SHBG_PHI	Number	Peak angle of friction (deg)
SHBG_RCOH	Number	Residual cohesion intercept (kN/m2)
SHBG_RPHI	Number	Residual angle of friction (deg)
CONS_REF	Autonumber	Number of consolidation test results within NETDATA
CONS_INCN	Number	Oedometer stress increment number
CONS_INCD	Text	Defined stress range (kN/m2)
CONS_INCF	Number	Stress at end of stress increment / decrement (kN/m2)
CONS_IVR	Number	Initial voids ratio
CONS_INCE	Number	Voids ratio at end of stress increment
CONS_INMV	Number	Coefficient of volume compressibilty over stress increment (m2/MN)
CONS_INCV	Number	Coefficient of consolidation over stress increment (m2/yr)

APPENDIX C

All publications – prior to 1996

PAPER C1 HORSLEY 1

PAPER C2 GENERAL (Q.M.)

PAPER C3 DEWATERING

PAPER C4 NORTHERN BACKFILL 1

PAPER C5 SITE INVESTIGATION

PAPER C6 DERWENT–TYNE CONFLUENCE

PAPER C7 GENERAL (B.G.)

PAPER C8 FAULTING – BUCKHEAD

PAPER C9 DRILLEX '87

PAPER C10 NORTHERN BACKFILL 2

PAPER C11 NORTHERN BACKFILL 3

PAPER C12 NORTHERN BACKFILL 4

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THE EFFECT OF A RISE OF WATER TABLE ON THE SETTLEMENT OF BACKFILL AT HORSLEY RESTORED OPENCAST COAL MINING SITE, 1973-1983

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INTRODUCTION

An increasing number of sites affected by opencast mining have been used in recent years for housing and industrial developments. This has led to considerable interest in the settlement of opencast mining backfills. The settlement of the ground surface has been monitored at a number of sites (Knipe, 1979; Leigh and Rainbow, 1979) and in some cases where construction has taken place, the settlement of buildings has been observed (Penman and Godwin; 1974; Charles et al, 1978; Gilbert and Knipe, 1979). An early investigation by Kilkenny (1968) suggested that the rate of settlement decreased rapidly with time and, for relatively shallow backfills, would often become negligible a few years after backfilling had been completed. However it has been found that on occasions large settlements can occur many years after the completion of backfilling. It was suspected that an increase in moisture content due to a rising ground water table or the ingress of surface water could often be the cause of such movements. It is well known that many uncompacted fill materials undergo a 'collapse' compression when saturated.

In 1973 an opportunity arose to investigate the effect of a rising ground water table on the settlement of an opencast coal mining backfill at Horsley in Northumberland. The backfill was up to 70 metres deep and composed largely of mudstone and sandstone fragments. Pumping had continued

after backfilling was completed and had kept the water table generally below the level of the fill. When pumping was stopped it was expected that the water level would rise rapidly through the backfill and thus present an ideal opportunity to monitor the ground movements caused by saturation of the backfill.

Instrumentation to measure settlement at various depths within the backfill was installed in 1973. Pumping stopped at the site early in 1974 and the water table rose rapidly reaching a new equilibrium level after three years. Settlement has been monitored before, during and subsequent to the rise in ground water level. The effect of the rising water table on ground movements has been assessed. A preliminary report on the measurements made up to the end of 1976 was presented to the First Conference on Large Ground Movements and Structures (Charles et al 1977).

HORSLEY SITE

The opencast workings covered an area approximately 1500 metres long by 600 metres wide and the site of the workings was close to Horsley village in Northumberland. The backfill is almost 70 metres deep in places. The excavated strata belong to the Middle and Lower Coal Measures of the Carboniferous system. When prospected, the site was found to be very wet with artesian water coming from boreholes passing below the pavement seam and from faults. In the upper part of the workings excavation of the overburden was carried out by face shovels and backfilling was by end tipping from dump trucks. In the lower part of the workings excavation was by dragline. The method of backfilling by end tipping leads to some segregation of the fill as the larger material tends to roll to the bottom of the tip. This is not so evident with dragline cast spoil. Part of the site was preloaded by a large overburden heap (Figure 1(a)) and another area was the site of a coal washing plant and lagoon (Figure 1(b)). Backfilling of different parts of the site took place over a ten year period and consequently there are considerable variations in the age of the backfill. Clearly there were many facets of the opencast mining operations which could give rise to considerable variations within the backfill.

The boreholes drilled in 1973 for the installation of the instrumentation yielded additional information about the backfill. The location of the boreholes is shown in Figure 2. 100 mm diameter open drive samples were taken at 10 metre vertical intervals in the boreholes. The samples showed a predominantly cohesionless fill of mudstone and sandstone fragments. There was an average content of silt size particles of about 10%; porosities ranged from 20% to 40%; the degree of saturation varied between 10% and 100%.

An investigation carried out by Durham University suggested that the ratio of weak rock debris (shale/ mudstone, seatearth) to strong rock (sandstone/siltstone) lay between 2:1 and 3:1 (Attewell, 1977). At the site of the lagoon, samples of wet cohesive fill were found. At gauge B2 close to the pump a 17 metre deep layer of boulders was found at 30 metres below ground level. These boulders had been placed around the pump during backfilling. An analysis of all the borehole logs suggested that under 10% of the backfill was boulders. Cavities up to 0.5 m deep were found. In general there appeared to be little systematic variation with depth or position of a borehole. However the samples did confirm that most of the fill was in a loose condition with a low degree of saturation.

Backfilling of different parts of the site took place between 1961 and 1970. Restoration of the experimental areas of the site to agreed surface contours, including replacement of subsoil and topsoil, was completed in 1973. A period of land management by the Ministry of Agriculture, Fisheries and Food followed. The land was released to the present occupiers in 1982. During the period of management the land was cultivated and sown down to grass and let for grazing and forage crops. Hedges and shelter belts have been planted, fences erected, underdrainage and field water supplies installed. The present use of the land is still mainly grazing and grass crops, with the probability of some cereal crops in the future.

During opencast mining it was necessary to de-water the site. Pumping continued for some time after the completion of backfilling to drain the adjacent Spital site. The location of the pump is shown in Figure 2. A field test carried out in a borehole at the site of the lagoon indicated a permeability greater than 10^{-4} metres/sec for the backfill.

INSTRUMENTATION

Late in 1973 five magnet extensometers were installed in 0.15 metre diameter boreholes drilled through the backfill using a Boyles Buccaneer rotary air flush rig. Details of the installation have been given by Charles et al (1977). Ring magnets were anchored to the sides of boreholes by strong springs at 6 metre vertical intervals. Their position could be detected subsequently by lowering a reed switch sensor with a steel tape attached down a central rigid access tube which was isolated from the settling backfill by an outer helically reinforced flexible tube. The bottom magnet of each gauge was installed in bedrock and formed a stable reference point. From measurements taken on successive occasions, settlement of the magnets relative to the reference magnet could be computed. Thus, at each

gauge, settlement could be monitored at different depths within the backfill.

The locations of the five borehole gauges were selected to provide as much information as possible about the behaviour of the backfill. Gauge A9 was installed in the oldest backfill on the site (1961). Gauge D15 was installed in the most recent (1970). Gauge B2 was located in the area of deepest fill, close to the pump. Gauge D1 was installed where the fill had been preloaded by an overburden heap with a maximum height above restored ground level of 30 metres and gauge C11 was installed in the old lagoon area. Information about the gauges is summarised in Table I.

A traverse of surface settlement stations was established adjacent to each borehole gauge to supplement the information on surface settlement. Precise levelling has been carried out from bench marks established on undisturbed ground outside the limits of the opencast workings. A plan of the site, with the borehole gauges and surface settlement traverses marked on it, is given in Figure 2. One traverse of surface settlement stations, traverse E, had no borehole settlement gauge adjacent to it. The fill in this area was of intermediate age.

TABLE I BOREHOLE SETTLEMENT GAUGES - BASIC DATA

Borehole settlement gauge	Ground level	Level of rockhead	Depth of backfill	Date of backfilling
	metres AOD	metres AOD	metres	
A9	98.6	38.0	60.6	1961
B2	101.8	38.7	63.1	1964
C11	94.9	49.2	45.7	1965
D1	108.1	52.6	55.5	1966
D15	119.2	72.7	46.5	1970

FIELD OBSERVATIONS

The five borehole settlement gauges were installed in 1973. Gauge D15 was installed first and measurements began in August 1973. The other four gauges were installed in November 1973. Precise levelling of surface settlement stations began in October 1973. Monitoring has continued until the present time, a period of ten years.

Pumping at the Horsley site had finally stopped by April 1974 and during the summer of 1974 the adjoining Spital opencast coal mine was backfilled and pumping stopped there also. The level of the water table at gauge

B2, installed in the deepest fill close to the pump, rose by 20 metres between April 1974 and April 1975, 9 metres in the following 12 months and 5 metres in the 12 months after that. From June 1975 onwards the water level measured in the five borehole gauges has been virtually the same height above OD. Having reached a new equilibrium water level in April 1977 at approximately 83 metres AOD subsequent fluctuations in water level have been small. A maximum water level of 84 metres AOD was recorded in August 1978. The final equilibrium level of the water table in the opencast backfill appears to have been controlled largely by the topography of the site. Although the Horsley opencast workings were separated from the Spital extension by a public road, this was temporarily diverted during opencast mining. Consequently, the highly permeable backfill is continuous between the Horsley opencast mine and the Spital extension. The ground generally slopes downwards from east to west and in 1978/79 a spring appeared on the western boundary of the Spital extension near the Whittle Burn. The ground level at the spring is 81.4 m AOD.

It is helpful when examining the settlement of the backfill to consider three periods.

(i) The few months prior to April 1974 when pumping kept the water table down in the bedrock below the backfill.

(ii) The three year period from April 1974 to April 1977 when the water table rose some 34 metres through the backfill.

(iii) Six and a half years subsequent to April 1977 during which the water table has shown only minor fluctuations.

Settlements recorded in the deepest part of the backfill at gauge B2 are plotted in Figure 3. At this gauge close to the pump the water table varied between 5 and 10 metres above rockhead during the initial period (i) in which pumping continued. The settlements recorded at ground level (magnet 13) were 8 mm during the four months of period (i), 338 mm during the 3 years of period (ii) as the water table rose 34 metres, 142 mm during the 6½ years of period (iii). The backfill is 63 metres deep but most of the settlement has occurred between the level of magnet 11, 8 metres below ground level, and magnet 5, 43 metres below ground level. By 1980 the rate of settlement had become very small and in view of this and for ease of presentation the timescale for the years 1981 to 1983 in Figure 3 has been changed. The same change in timescale has been adopted for Figure 4 in which are plotted the vertical compressions measured between adjacent magnets at gauge B2. The effect of the rising water table becomes

quite clear when Figure 4 is examined. As the water table rose from the level of magnet 5 to the level of magnet 6 a vertical compression of just over 1% occurred over the depth of fill between these two magnets. As the water table continued to rise large compressions occurred successively between magnets 7 and 8, 8 and 9, and, 9 and 10. The effect of saturation in producing collapse compression within the backfill is thus clearly demonstrated. Locally compressions as large as 2% were recorded but the average compression over the 34 metre depth of backfill saturated by the rising water table was smaller than 1%. Although the relationship between the rising water table and settlement of the backfill is very clear, it should be noted that, at an early stage a small expansion occurred between magnets 4 and 5, little compression occurred between magnets 6 and 7, and throughout the 10 years of monitoring, significant compression has occurred between magnets 10 and 11 which are well above the final equilibrium level of the water table.

At gauge A9 the borehole caved in during installation of the magnet extensometer which as a result does not extend down through the full depth of the backfill and has no reference magnet installed in bedrock. Nevertheless the vertical compression between adjacent pairs of magnets can be examined and some correlation with the rising water table is again evident. This was most marked as the water table rose from the level of magnet 3, 36 metres below ground level, to magnet 4, 29 metres below ground level, when a vertical compression of 1.4% occurred between the two magnets.

Settlements measured at gauge D1 are plotted in Figure 5 and the vertical compressions between adjacent magnets are presented in Figure 6. The backfill in this locality had previously been loaded by a large overburden heap with a maximum height above restored ground level of about 30 metres (Figure 1(a)). This had been removed 2 years before the measurements began late in 1973. The water table remained below rockhead until pumping stopped. Four months of monitoring prior to April 1974 in period (i) showed a heave of 14 mm at ground level (Figure 5). During the subsequent 3 year period (ii), the water table rose and saturated the bottom 30 metres of the backfill at this location. Figure 6 shows that as the water table rose from the level of magnet 2 to the level of magnet 3, a small vertical expansion occurred in the backfill between these two levels. As the water table continued to rise, vertical compressions occurred successively between magnets 3 and 4, 4 and 5, 5 and 6. It should be noted that, firstly a further rise in water table above magnet 6 caused no compression between magnets 6 and 7; secondly most of the settlement observed at gauge D1 was located more than 28

metres below ground level; thirdly at depths where compression was caused by the rising water table the magnitude was much smaller than at gauge B2. All these three observations can be attributed to the effect of pre-loading with the overburden heap.

The settlements measured at gauge C11 are plotted in Figure 7. The water table had risen from below rockhead to 34 metres above it by April 1977. In general the settlement shows little indication of being affected by the rising water table. As this area was the site of a lagoon during opencast working (Figure 1(b)) it is probable that the backfill was sufficiently wetted at that stage to prevent further settlement occurring due to the rising water table. Figure 7 indicates that only when the water table rose from the level of magnet 5 to the level of magnet 6 did compression occur that was clearly associated with the rise in water level. In the four months of monitoring during period (i) the settlement at ground level (magnet 10) was 18 mm. During the following three years of period (ii), 86 mm settlement occurred. Movements during period (iii) have been negligible. At this gauge settlement occurred fairly uniformly through the full depth of the backfill.

The rising ground water table had little effect on settlement at gauge D15 because the gauge is situated on high ground (Table I) and only the bottom 10 metres of the backfill have been inundated. This is the most recently placed backfill and currently the rate of settlement is greater than at any of the other borehole gauges. Measurements at this gauge taken between August 1973 and April 1975 give an interesting picture of creep in the opencast backfill unaffected by changes in water level. Figure 8 shows that in this 20 month period the water table was below rockhead, settlement at ground level (magnet 10) was 37 mm, at 7 metres below ground level (magnet 9) it was 16 mm and at 26 metres below ground level (magnet 6) no settlement had occurred. Between April 1975 and April 1977 as the water table rose and inundated the bottom 10 metres of the backfill, there was a significant increase in the rate of settlement. Most of the settlement at this stage occurred towards the bottom of the fill. During this 24 month period the additional settlement at ground level was 97 mm, at 7 metres depth it was 72 mm and 9 metres above bedrock it was 24 mm. During the following six and a half years the settlement at ground level has been 118 mm, at 7 metres depth 78 mm, and 9 metres above rockhead only 4 mm. It is seen that the pattern of settlement prior to and subsequent to the rise of the water table was quite similar. A significant proportion of the settlement was located in the upper 7 metres of the backfill. In contrast, whilst the water table was rising, the increased rate of settlement was

largely due to compression occurring at the bottom of the backfill.

The surface settlements monitored at the five borehole gauges are plotted in Figure 9. The differences in the settlement behaviour observed at the five gauges are readily apparent. The smallest movements have occurred at gauge D1 and this can be attributed to pre-loading by the overburden heap. The settlement at gauge C11 has also been small but it should be noted that prior to the rise in water level the settlement rate at this gauge was greater than in most of the other parts of the site. It may well be that in the years before monitoring commenced the settlement of the lagoon area, which is composed of a wet and more cohesive fill, was very large. Currently the greatest rate of settlement is occurring at D15 which is the most recently placed backfill and is on high ground little affected by the rising water table. The largest surface settlement recorded at the borehole gauges over the 10 year monitoring period has occurred at gauge B2 where the fill is deepest. Nearly 0.5 metres of settlement has been measured there.

Surface settlements measured by precise levelling along the traverses between October 1973 and October 1983 are plotted in Figure 10. At traverse B the settlement profile is similar to the topographical profile, ie the settlement is roughly proportional to the depth of the backfill and equal to nearly 1% of the fill depth. No such correlation between settlement and depth of fill is found at traverse D. Two other factors may have had a major influence on settlement in this region; firstly the pre-loading of the backfill in the vicinity of gauge D1 by the overburden heap, secondly the differences in age of the backfill. The largest recorded settlements have occurred on traverse A where a movement of 0.65 metres has been measured corresponding to an average vertical compression of 1.1% of the depth of backfill and on traverse E where 0.72 metres settlement corresponds to 1.5% compression. At traverse C settlements were small but it has already been pointed out that prior to the commencement of monitoring in 1973 large settlements may well have occurred in this old lagoon area. Quite large settlements have been monitored at station 13 on traverse A (186 mm) only a few metres away from the edge of the backfill. The largest differential settlements away from the edge of the workings of stations 5 metres apart are 69 mm on traverse A and 89 mm on traverse E.

DISCUSSION

The Horsley investigation has shown that the pattern of settlement of a restored opencast mining site can be complex. Even with a good knowledge of the opencast operations that had occurred on the site it could be quite

difficult to predict the magnitude and period of the settlement of the backfill. However the investigation has clearly shown the relationship between some of the features of the opencast mining and subsequent settlement behaviour.

The dewatering of the site during opencast operations made it virtually certain that at some stage much of the backfill of the restored site would be inundated by a rising water table. Indeed the major concern in this investigation was to establish whether or not a rise in water table would cause collapse compression of the backfill. A correlation between settlement and the rise in water table has been clearly established. Vertical compressions of up to 2% were measured locally, but the average compression over the full depth of saturated backfill was smaller than 1% at the borehole gauge locations.

The behaviour monitored at gauge D1 (Figures 5 and 6) has shown the major effect on settlement characteristics that temporarily pre-loading the backfill with an overburden heap can produce. Collapse compression when the backfill was saturated was significantly smaller than at gauge B2 and during the 10 years of monitoring virtually no compression has occurred in the upper 28 metres of the backfill. Prior to the rise in water table, the ground surface was heaving at this location.

Gauge C11 was installed in an area that had been a lagoon during opencast operations. Some of the fill here was more cohesive and was generally wetter than in the rest of the site. Pre-wetting of the backfill meant that the rising water table had little effect on settlement (Figure 7). During the ten years of monitoring, settlements have been small. However in the early stages before the water table rose, the settlement rate at this gauge was large and it may be that in the years immediately following backfilling some of the largest settlements occurred in this area.

The significance of the age of backfill is illustrated by the settlements monitored at gauge D15 (Figure 8). This is in the most recent fill and currently the settlement rate being monitored here is much greater than at the other gauges.

Although a relationship between the rising water table and settlement has been clearly established, it should not be concluded that this was the sole cause of settlement of the backfill. At gauge B2 25% of the settlement was caused by vertical compression of the unsaturated backfill between 8 and 14 metres below ground level (Figure 3).

There are no plans for building developments on the Horsley site, so far as is known, but the usefulness of the investigation lies in the significance of the observed settlements for building developments on restored opencast mining sites. The complexity of the settlement pattern measured at Horsley points to the need for careful investigation of restored opencast mining sites. Two important component parts of such an investigation should be firstly to ascertain as much detail as possible about the opencast operations and secondly to monitor settlement and water levels within the backfill over a reasonable period of time. The results from the Horsley investigation have shown the potential problems that could be caused by an unsaturated opencast mining backfill being inundated subsequent to building on the site.

CONCLUSIONS

1 The complex pattern of settlements observed at the Horsley restored opencast mining site can largely be related to details of the opencast mining operations.

2 The rising ground water table caused significant settlement of the uncompacted backfill which was composed mainly of mudstone and sandstone fragments. Vertical compressions were locally as large as 2% but the average settlement measured over the full depth of inundated backfill at the borehole gauges was smaller than 1%.

3 A significant proportion of the settlement monitored at the borehole gauges was due to compression in the upper 10 metres of backfill which was not saturated by the rising water table.

4 Temporary pre-loading with a large surcharge of fill greatly reduced subsequent settlement due to the rising ground water table and virtually eliminated any long term creep settlement.

5 Pre-inundation at the site of the lagoon greatly reduced the effect of the rising water table on settlement. However the wet and more cohesive fill at this location may have suffered large settlements in the years immediately following backfilling.

6 Building developments on restored opencast mining sites should be preceded by careful investigations. As much information as possible should be obtained about opencast operations, and settlement and water levels should be monitored over a realistic period. In addition to settlement due to self-weight and applied loads, the possibility of collapse settlement on inundation should be considered.

ACKNOWLEDGEMENTS

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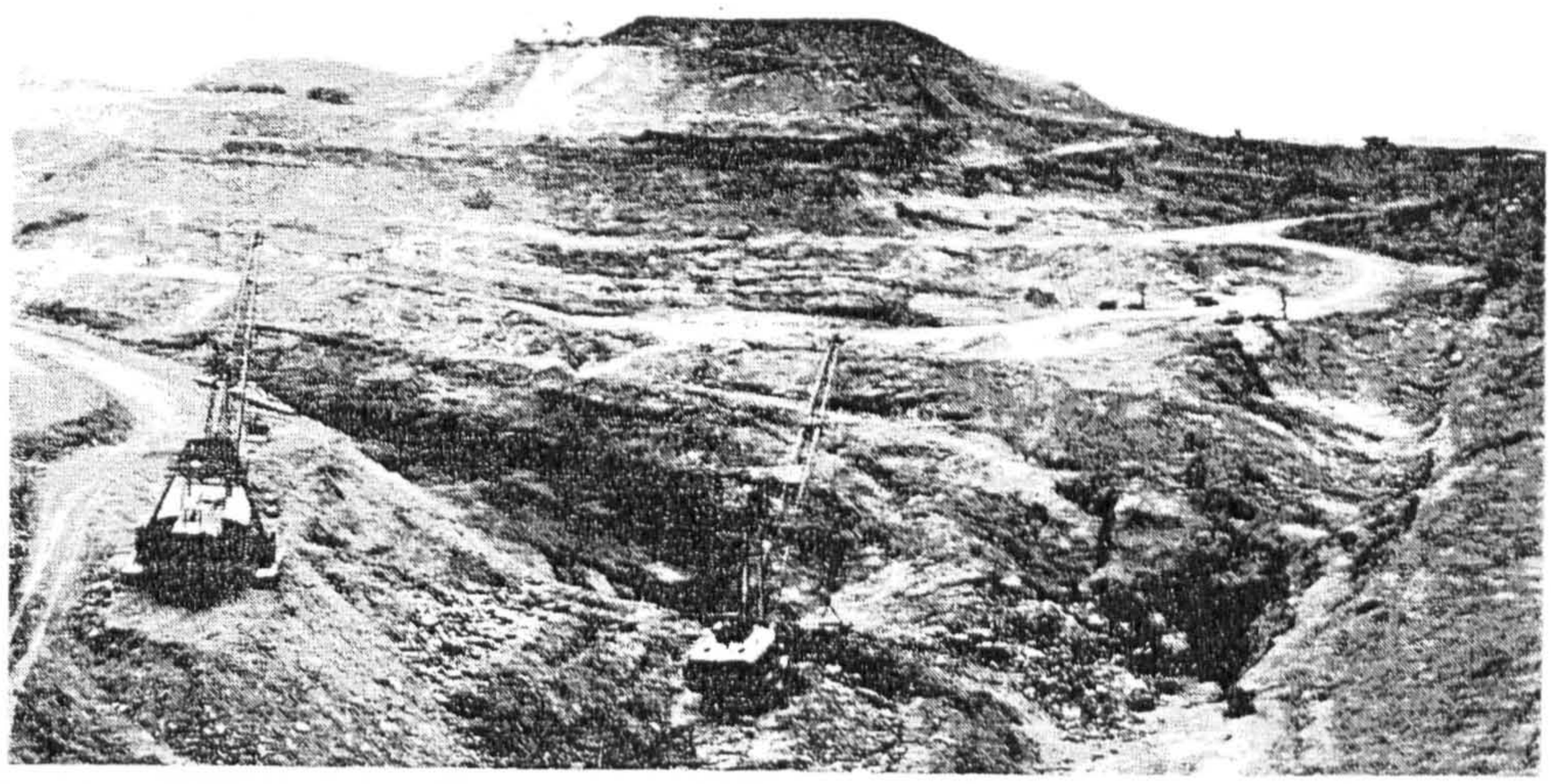
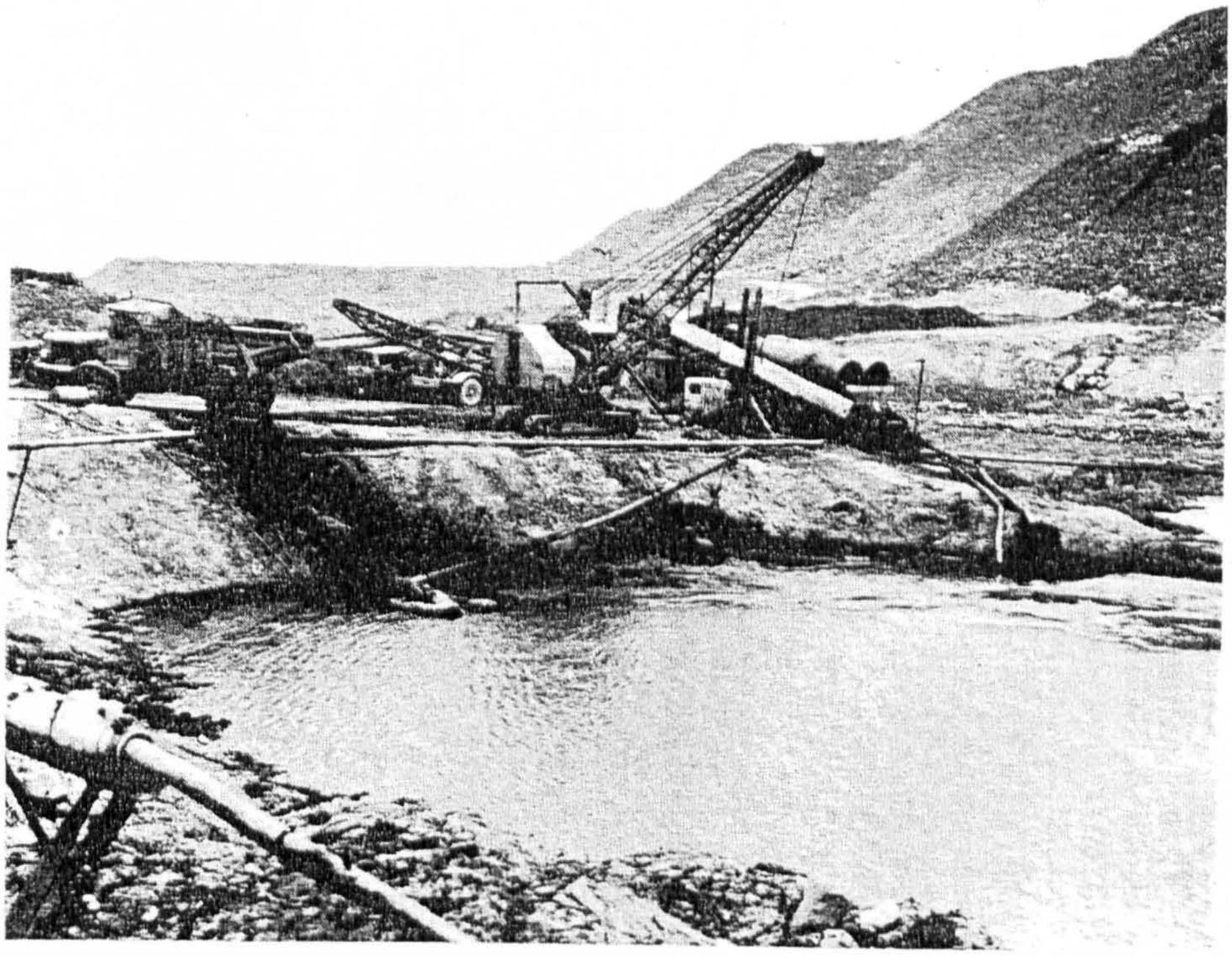


(a) Opencast mine and overburden heap



(b) Coal washing plant and lagoon

FIGURE 1 Horsley site during opencast mining

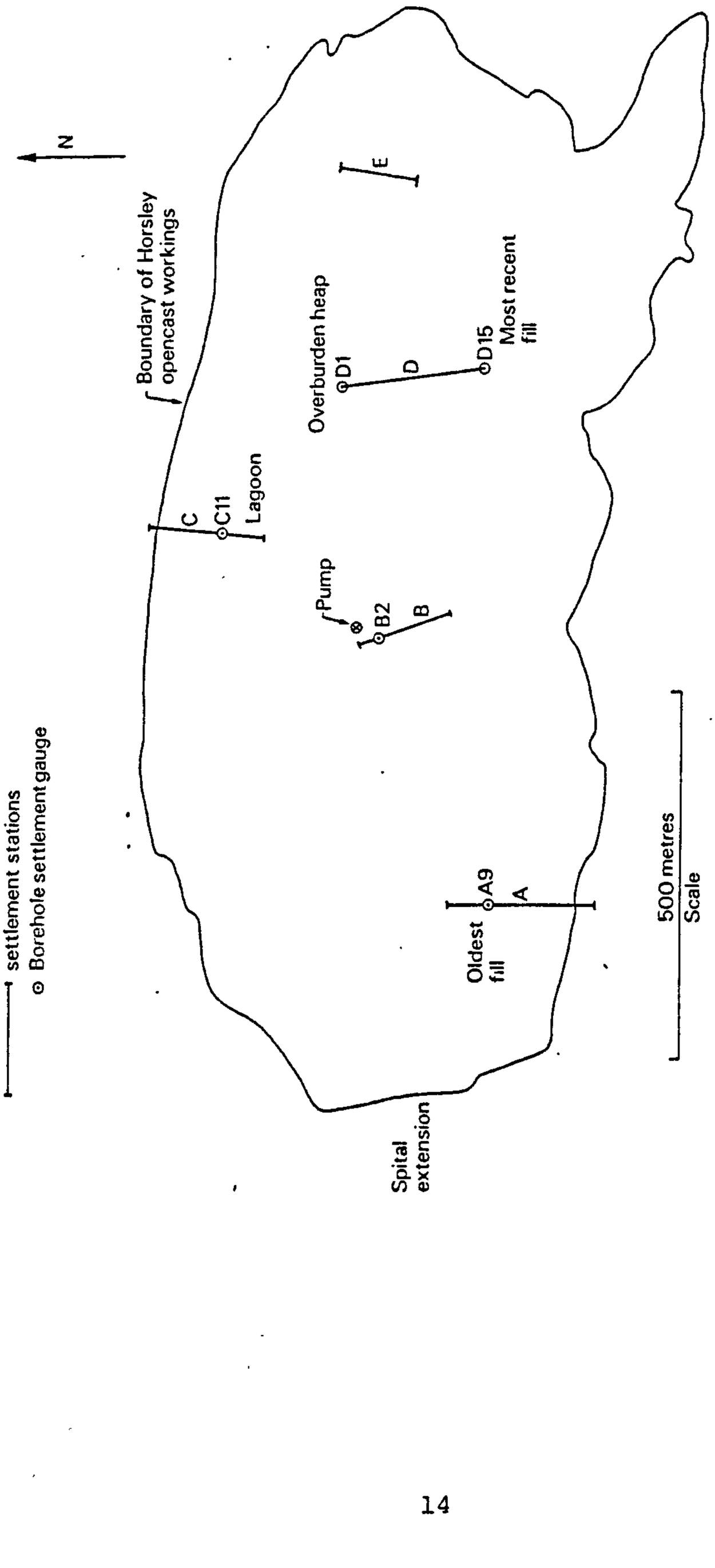


FIGURE 2 Plan of Horsley site showing location of instrumentation

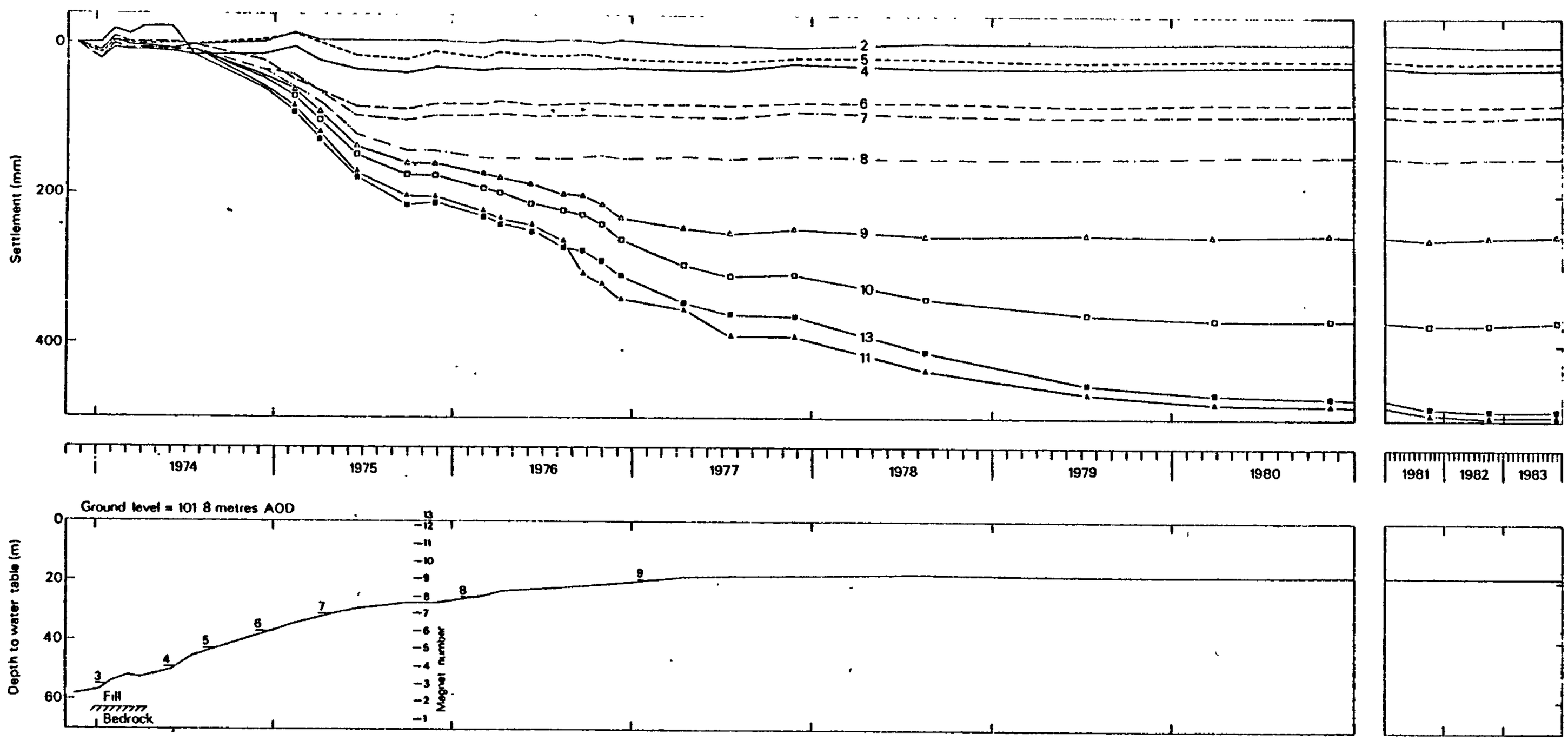


FIGURE 3 Settlement monitored at gauge B2 in deepest backfill

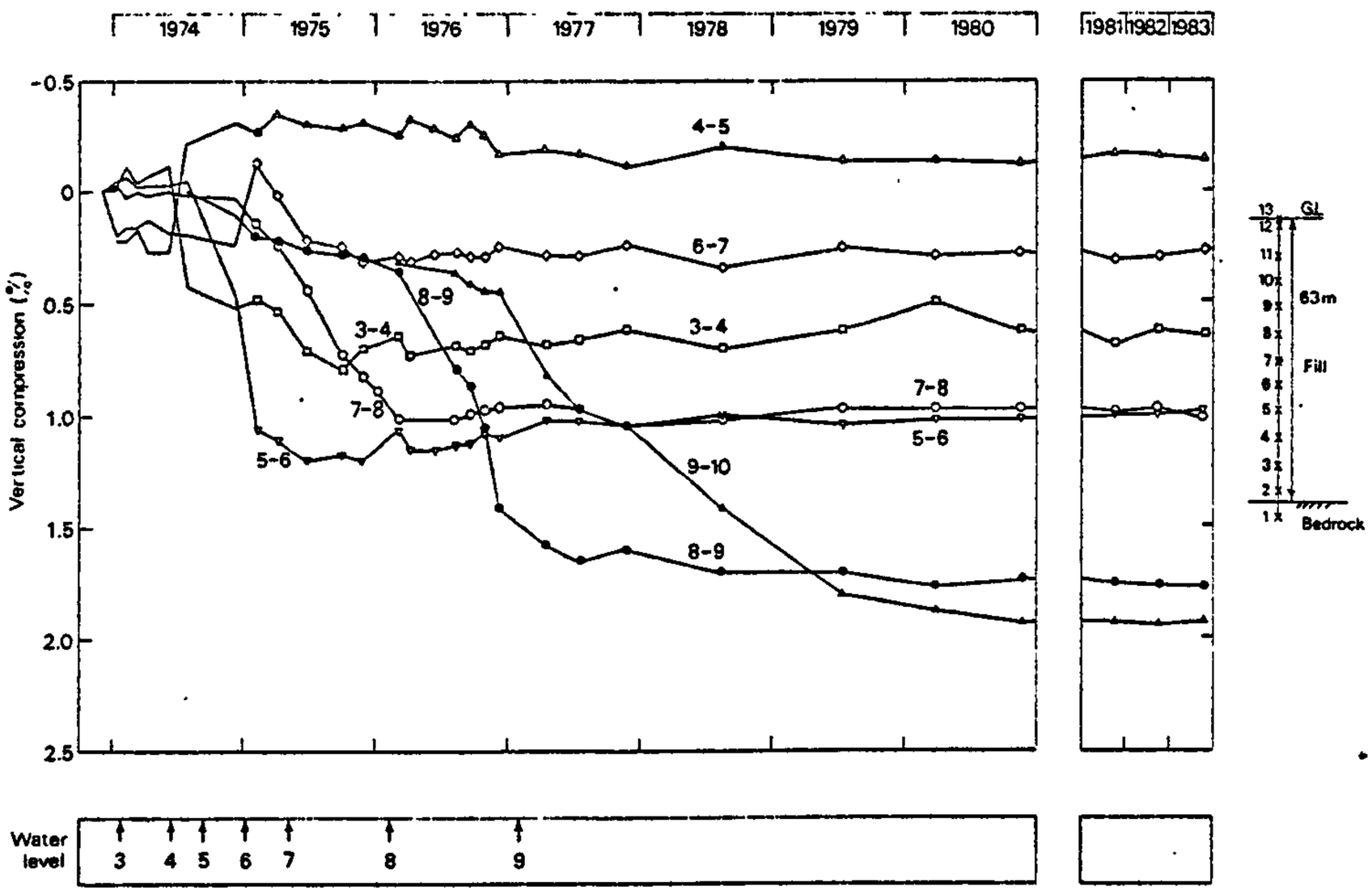


FIGURE 4 Vertical compression at gauge B2 in deepest backfill

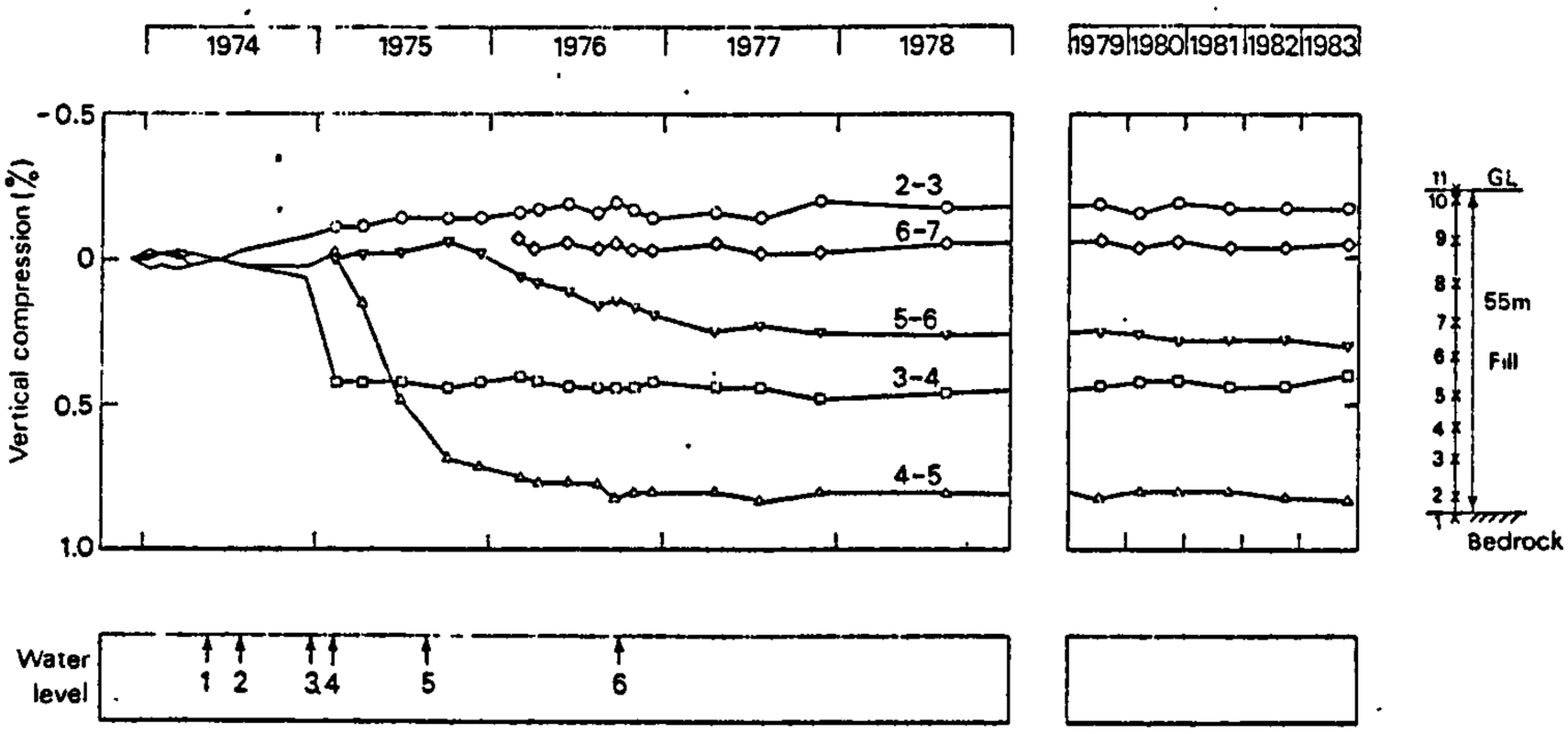


FIGURE 6 Vertical compression at gauge D1 where backfill had been preloaded by overburden heap

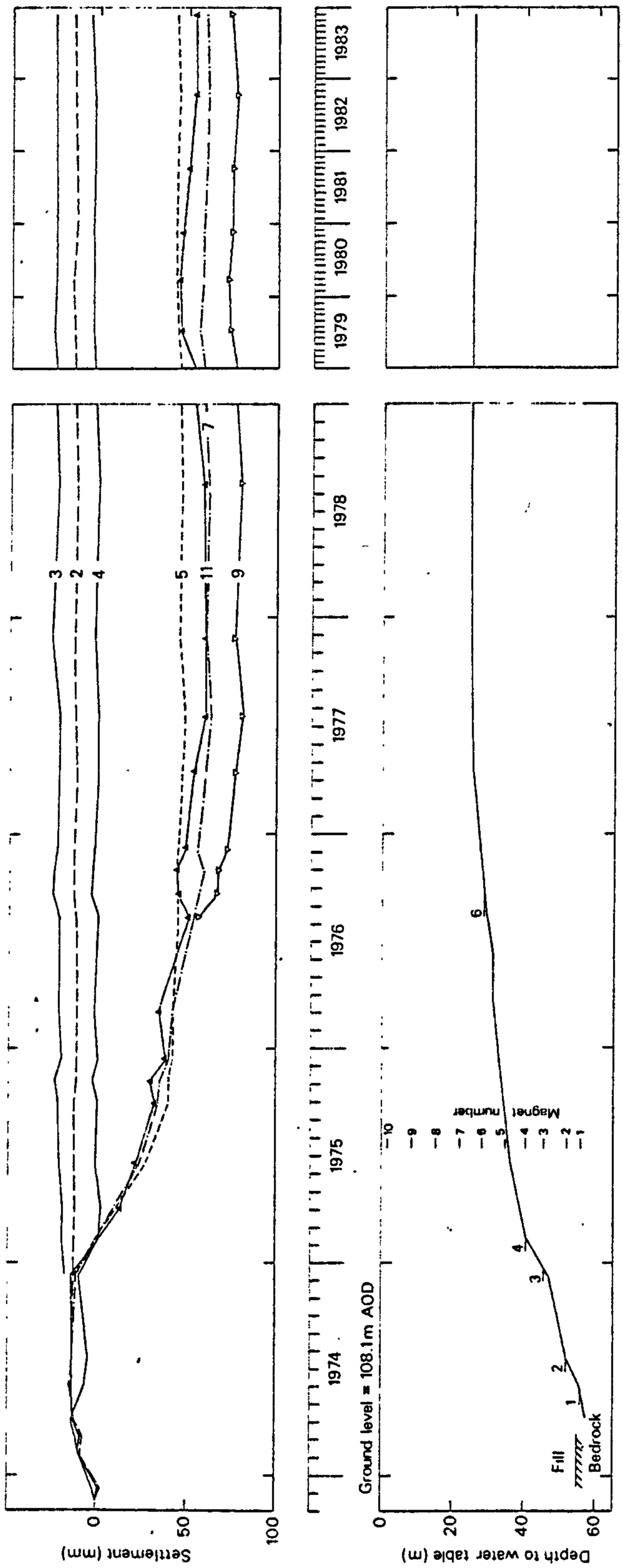


FIGURE 5 Settlement monitored at gauge D1 where backfill had been preloaded by overburden heap

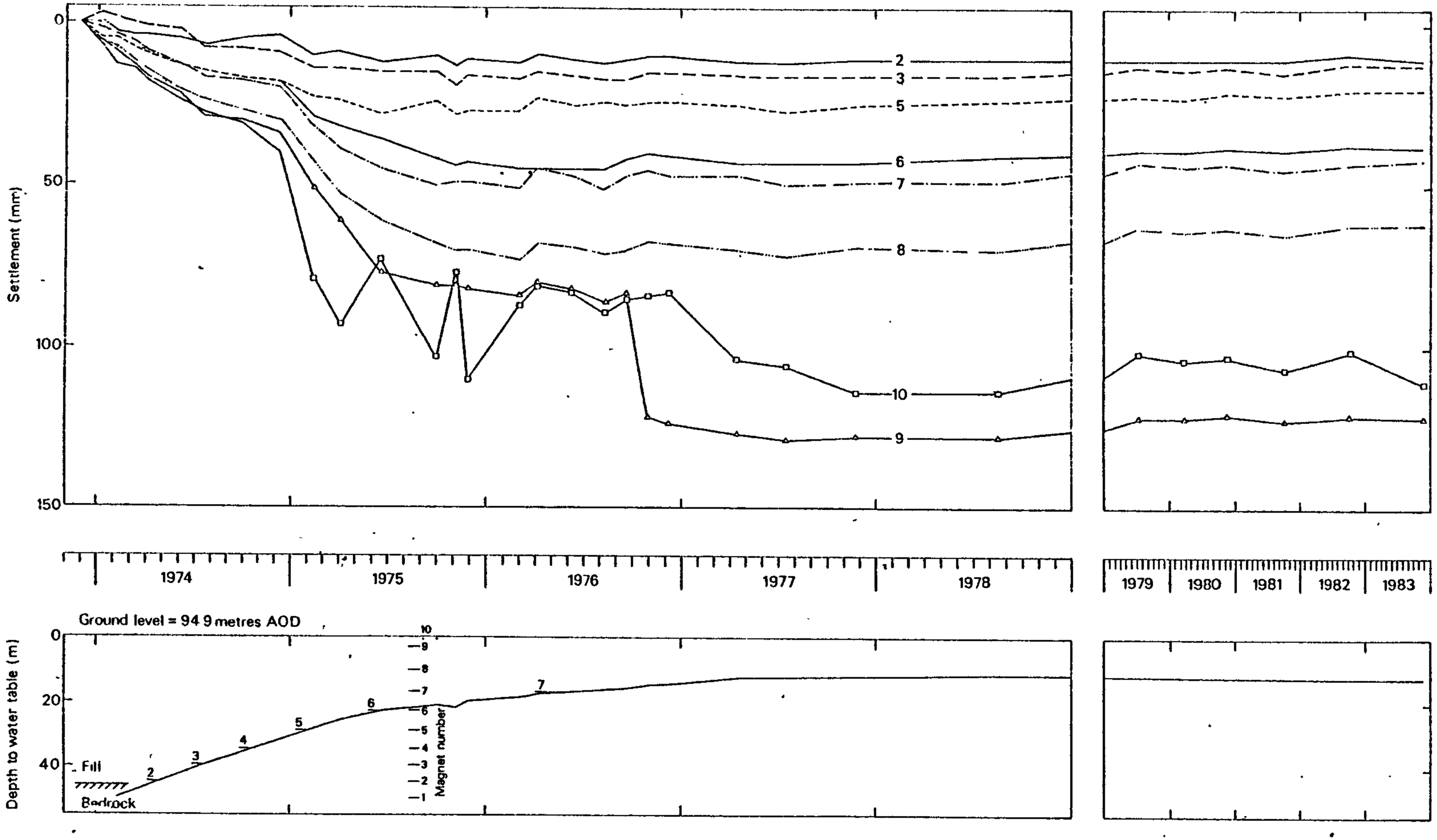


FIGURE 7 Settlement monitored at gauge C11 in former lagoon area

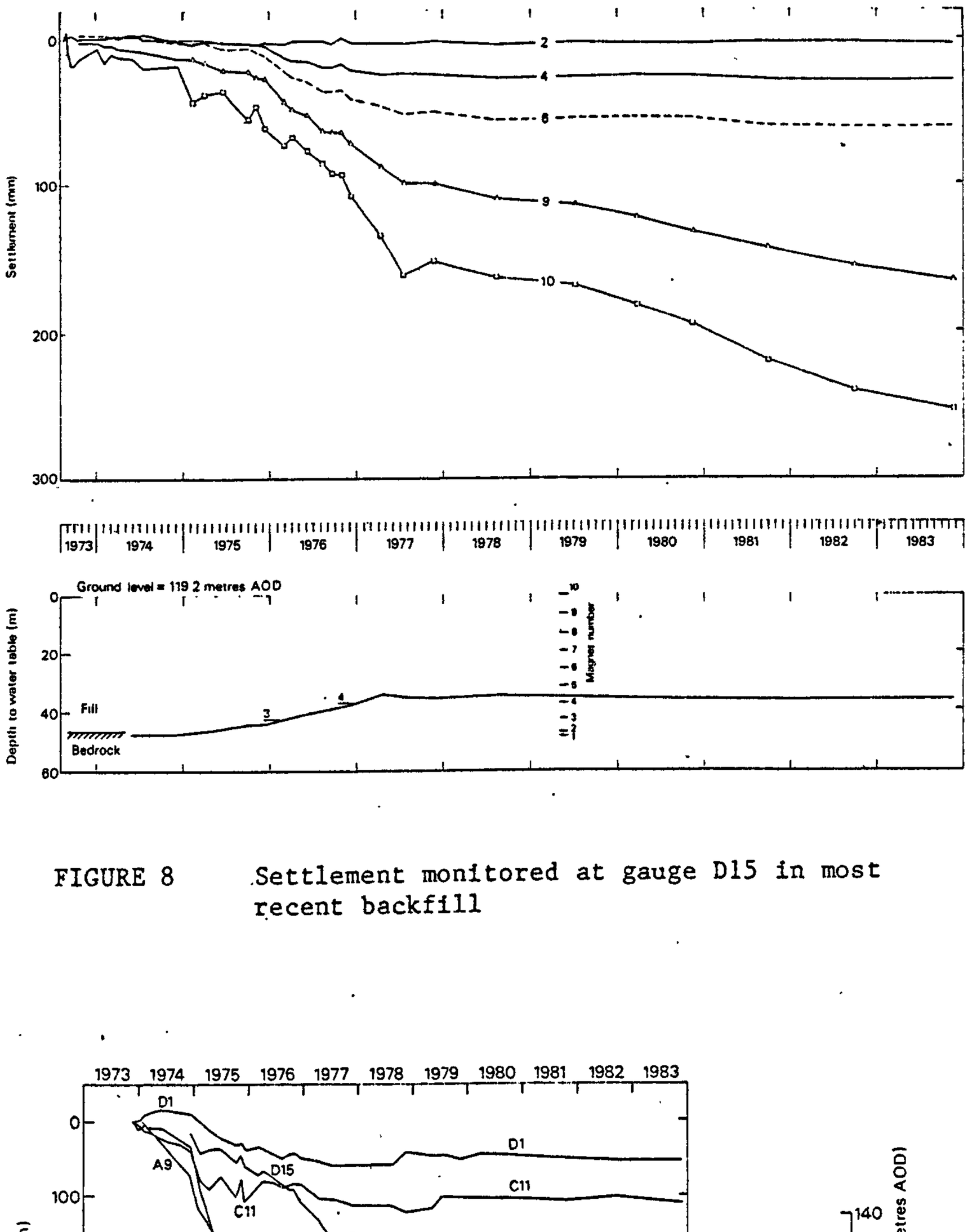


FIGURE 8 Settlement monitored at gauge D15 in most recent backfill

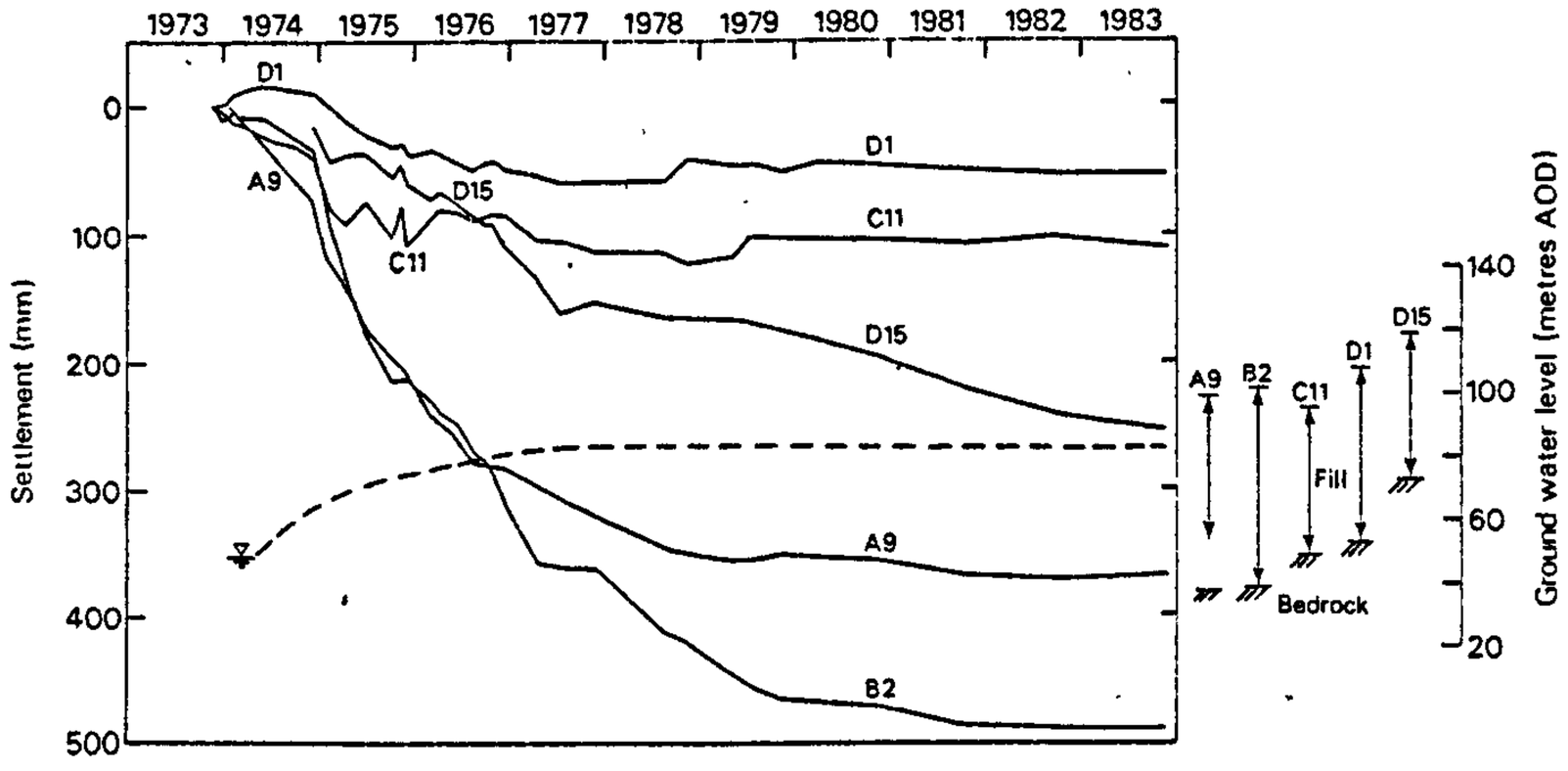


FIGURE 9 Surface settlement monitored at 5 borehole settlement gauges

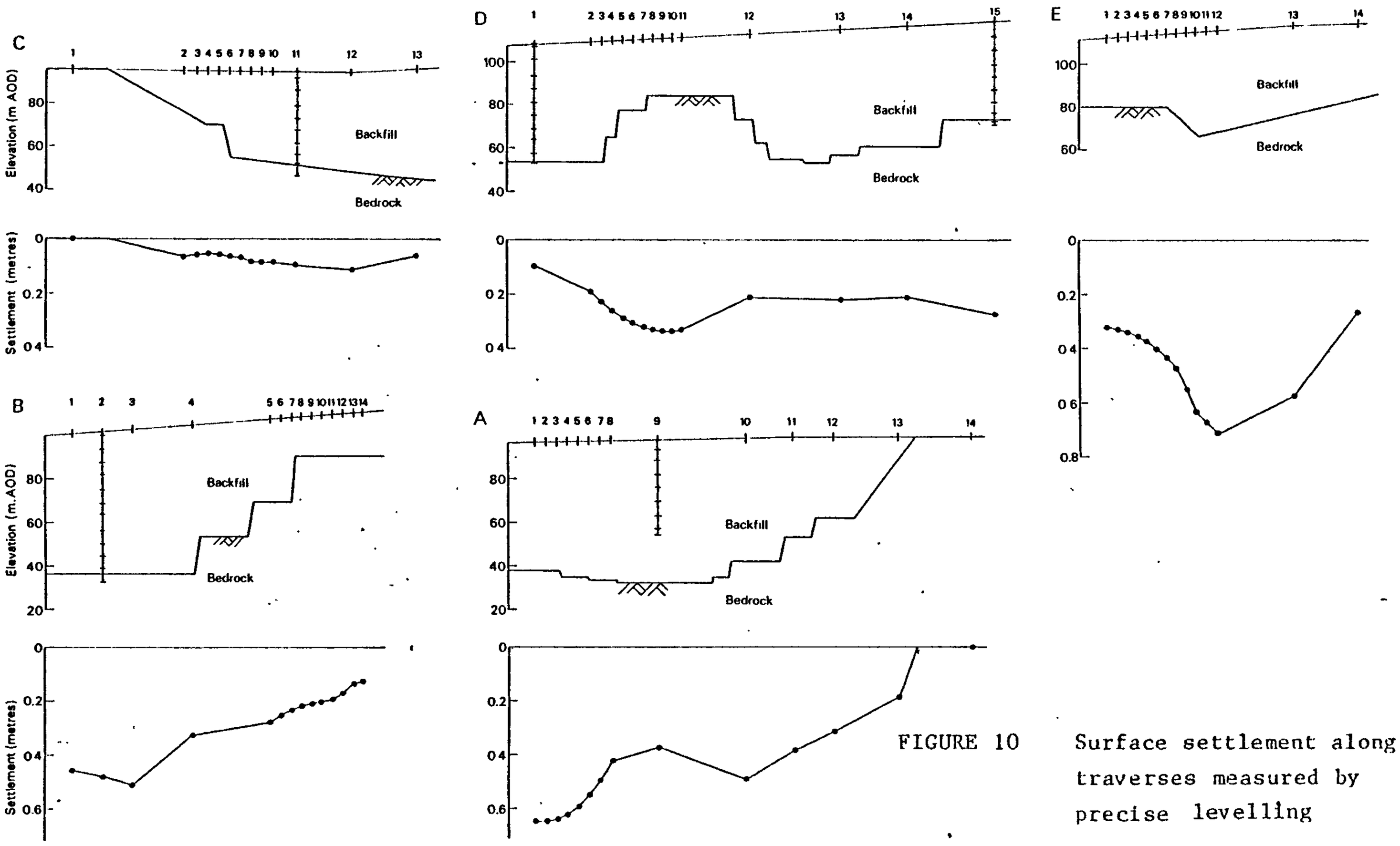


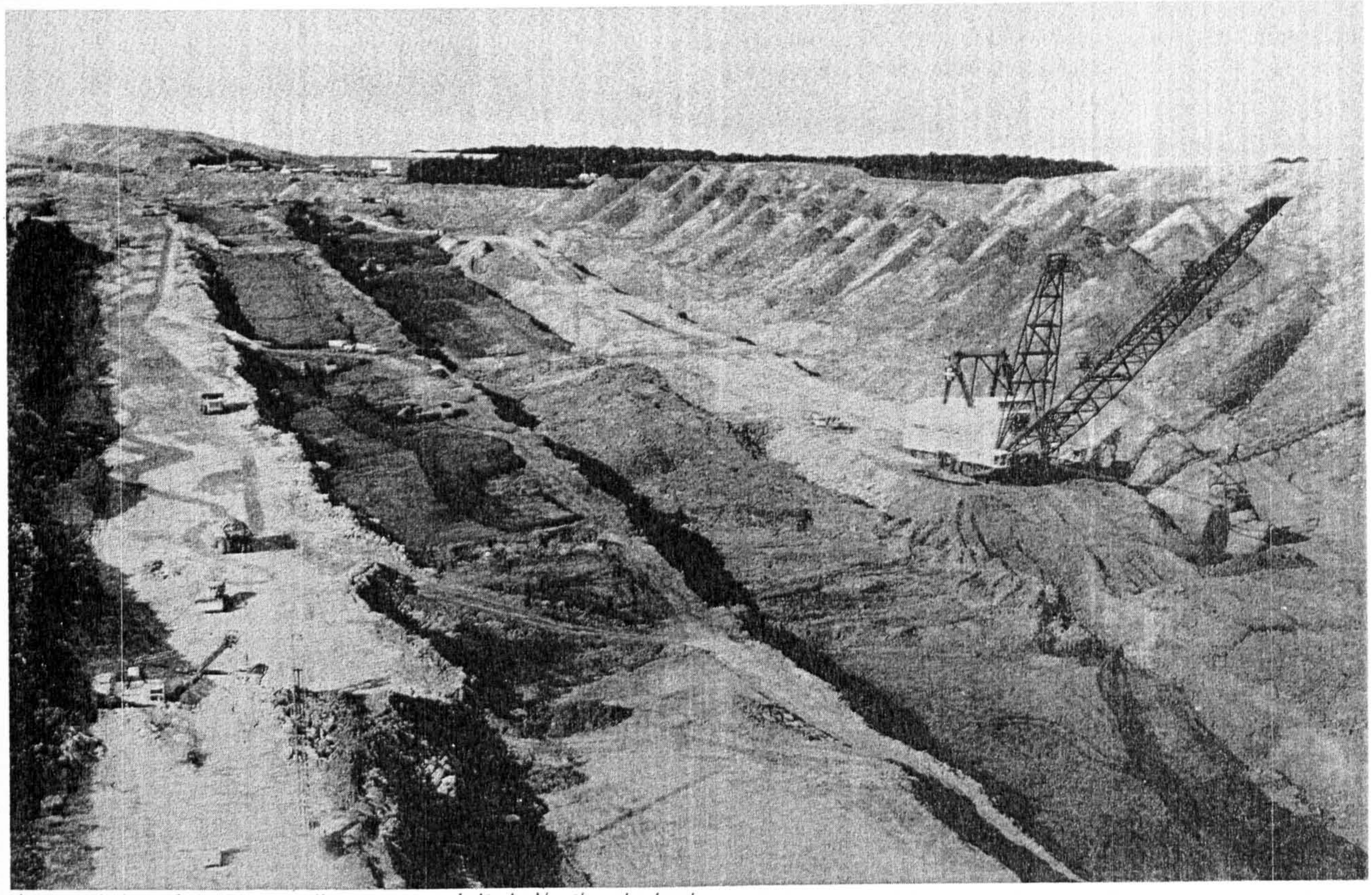
FIGURE 10 Surface settlement along traverses measured by precise levelling

PAPER C2 GENERAL (Q.M.)

Hughes, D.B. and Leigh, W.J.P. 1985.

The stability of excavations and spoil mounds in relation to opencast coal mining.

Institute of Quarrying Transactions (Quarry Management), April 1985, 223-229/232.



A general view of the Butterwell opencast coal site in Northumberland

THE STABILITY OF EXCAVATIONS AND SPOIL MOUNDS IN RELATION TO OPENCAST COAL MINING

by D. B. Hughes and W. J. P. Leigh

This paper was presented at a meeting of the North of England branch of the Institute of Quarrying on 24 September 1984

Annual coal production by the National Coal Board's Opencast Executive is currently between 14 and 15 million tonnes, which is obtained from about 60 different sites located in England, Scotland and Wales. The extraction of 1 tonne of coal may typically involve the removal of around 10–20m^3 of overburden. Average working depths are 30–40m, with 80–100m depth being quite common; only very exceptionally does the depth of excavation exceed 200m. Dependent on the quantity of coal available, most site areas range between 30 and 800 hectares, with varying proportions of overburden requiring to be stored above ground; in exceptional cases this quantity can be as much as 50 million m^3.

Opencast coal mining operations

The operations at an opencast coal mine typically involve excavating through superficial deposits and Coal Measures strata in order to win coal from one or more seams. First, topsoil and subsoil are removed and placed in mounds or dumps for storage. Then excavation to the pavement (basal) coal seam takes place in a series of cuts. The sequence of operations for single-seam working is shown in fig. 1.

In horizontal or gently dipping strata the initial cut or 'box cut' is usually excavated parallel to the strike direction of the seam, the spoil from this cut being placed in a suitable location on the ground surface to form an overburden mound. Once the coal has been extracted, the next cut is excavated and the spoil placed in the previous cut and so on. After the final cut has been excavated and the coal removed, the initial cut spoil (from the overburden mound) is placed into the final void. This operation is followed by grading, rooting and stone-picking of the replaced overburden, and then the subsoils and topsoils are put back. Figure 1 shows the effect of bulkage (ie volume increase) when excavated materials are replaced.

The authors are with the NCB Opencast Executive.

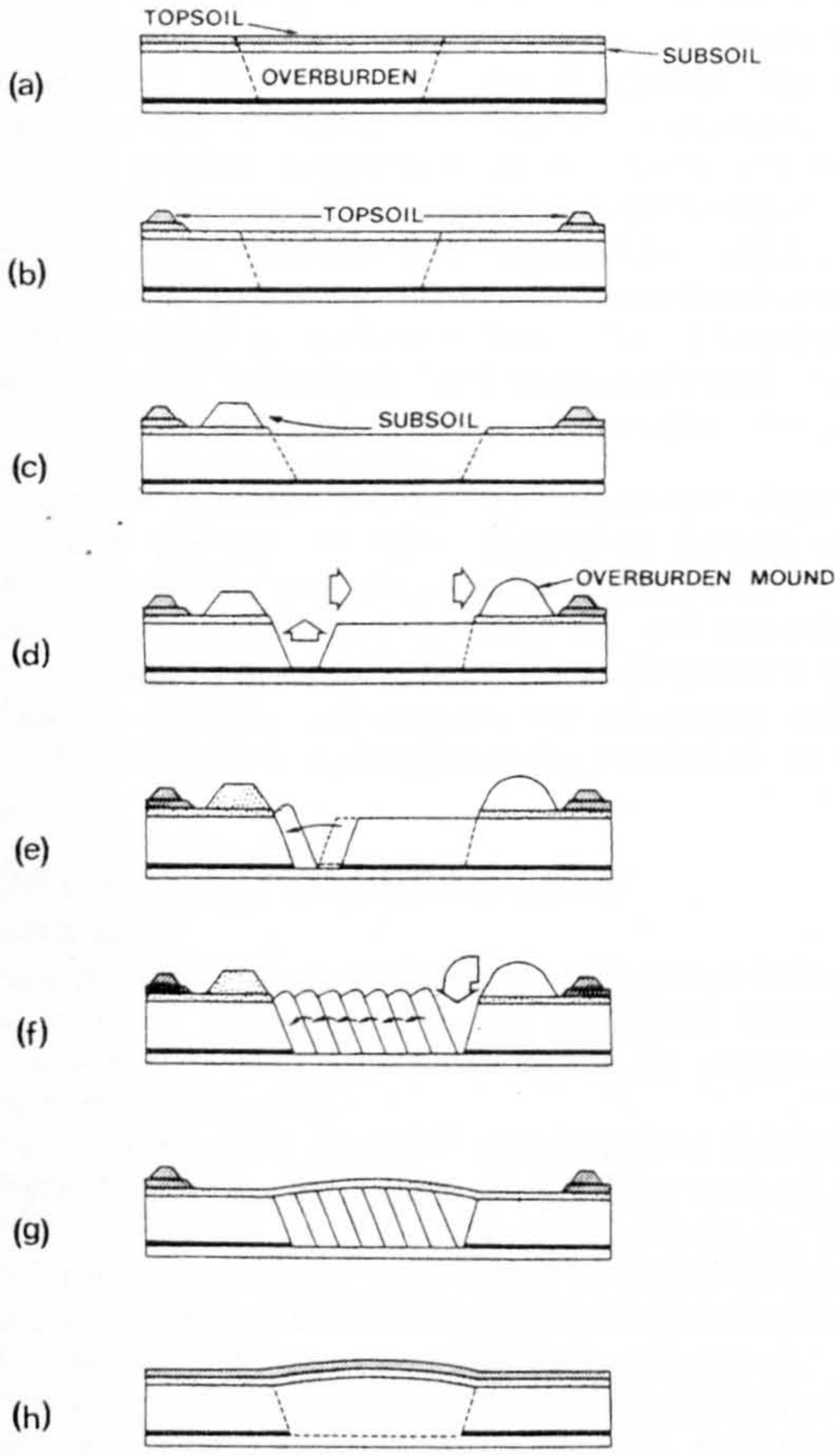


FIG. 1 Opencast coal mining – simplified sequence of working

Sometimes the dip of the seams is such as to involve the risk of backfill sliding into the cut or strata in the advance face of the cut (highwall) sliding down the bedding planes. In this case the cuts are usually excavated parallel to the full dip of the strata and the series of cuts proceeds in the strike direction.

In a few places in Britain the dips are so steep that cuts excavated parallel to the dip cannot be used because plant and vehicles cannot move on the steep dips. In these circumstances the open-pit method is adopted, which basically involves lowering the floor of the pit and working the steep seams as the pit descends. This method involves storing a larger quantity of overburden spoil in mounds above ground level than in the more usual strike-cut or dip-cut methods. Replacement of the backfill is again progressive, the area being worked in a series of 'panels', ie very wide cuts.

The types of excavation plant employed are normally tractor-scrapers, faceshovels and draglines. Frequently the working of a site involves the combination of all three types. Figure 2 shows a plan and section of a typical multi-seam operation, and serves to illustrate the terminology used to describe some of the main stability features in an opencast mine, eg highwall, endwall, loosewall, pavement, safety bench etc.

GEOLOGY

The general situation is as illustrated in fig. 2, ie Coal Measures rocks overlain by superficial deposits.

Coal Measures rocks

The principal rock types are sandstone, siltstone, mudstone, coal, seat earth, which occur in sedimentary layers and are subdivided by bedding planes. Other structural features include faults, joints, intraformational shear zones (clay mylonites) after Stimpson and Walton[1] and Salehy *et al.*[2]) and old underground workings which cause void migration and fracturing in the overlying strata.

Superficial deposits

Most opencast coal mining regions in Britain have some degree of superficial soil cover over the Coal Measures bedrock, but this varies considerably between the regions; for example, in the north of England about 25% of all excavated materials on opencast sites are classified as superficials and mainly comprise glacial tills (boulder clay, laminated clay, sand and gravel etc.) also some residual soils, alluvial deposits, peat and made ground (colliery spoil and opencast backfill etc.). In the midlands and Yorkshire there is only a very occasional cover of glacial material and the proportion of superficials is much less; but there are still some areas of residual soils, alluvium and made ground.

Other formations

Occasionally Permian or Triassic strata overlie the Coal Measures, eg red sandstones in Cumbria and red marls in Leicestershire. Also minor igneous intrusions are sometimes encountered, eg dykes in Northumberland and sills in Scotland.

Groundwater

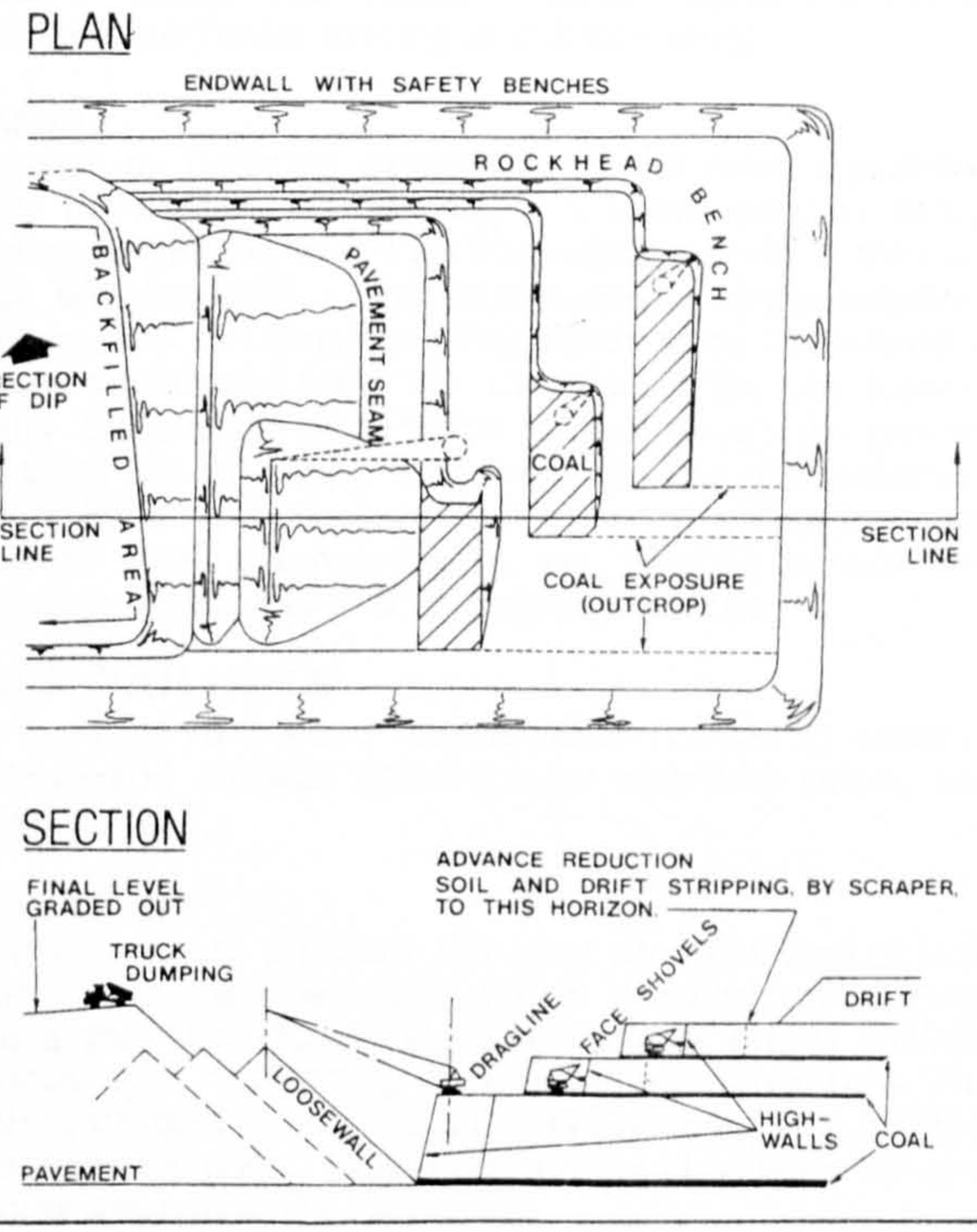
Groundwater is present within the Coal Measures bedrock at many sites, and may have a very significant bearing on stability problems. Sandstone beds and coal seams containing old workings usually act as aquifers. Mudstones and seat earths may be aquicludes.

Perched water tables may occur within the superficial materials, eg in old opencast backfill, or in sand and gravel lenses within the glacial drift.

PROSPECTING AND GEOTECHNICAL INVESTIGATIONS

Open-hole air-flush rotary drilling techniques are mainly used to prove the quantity of coal within a potential site, also to investigate the strata succession and depth of superficial deposits etc. The borehole spacing adopted is usually

FIG 2 Plan and section diagrams of a typical opencast mining operation



determined by the degree of structural complications and the occurrence of old underground workings. In a proportion of boreholes, core samples of the coal seams plus roof and floor measures are obtained for quality assessment and correlation. A smaller proportion of boreholes are core-sampled to their full depth and logged in engineering terms, as an aid to both stability and diggability assessment. Piezometers or standpipes are installed in some boreholes to permit monitoring of groundwater levels. Also a number of geophysical logging techniques have been developed which are used to provide supplementary information on coal seam and rock mass characteristic.

Geotechnical investigations of the superficial deposits usually require the use of cable percussion boring techniques. In situ testing and piezometer installation may be carried out. Also samples are obtained for soil mechanics laboratory testing. The results from these laboratory tests are applied to stability calculations for excavated slopes and spoil mounds when investigating the feasibility of site working.

FACTORS AFFECTING STABILITY

Rocks and soils

In studying the mechanisms of opencast mine slope failures, we must consider separately the two geological materials through which we make our excavations, ie engineering rocks and engineering soils.

Failures in rock tend to occur along existing discontinuities which dissect the rock mass (eg bedding, jointing and faulting) with little or no failure through the intact rock material. The resistance of rock mass to slope failure is governed by the orientation of the discontinuities and the shearing resistance which can be mobilized along them.

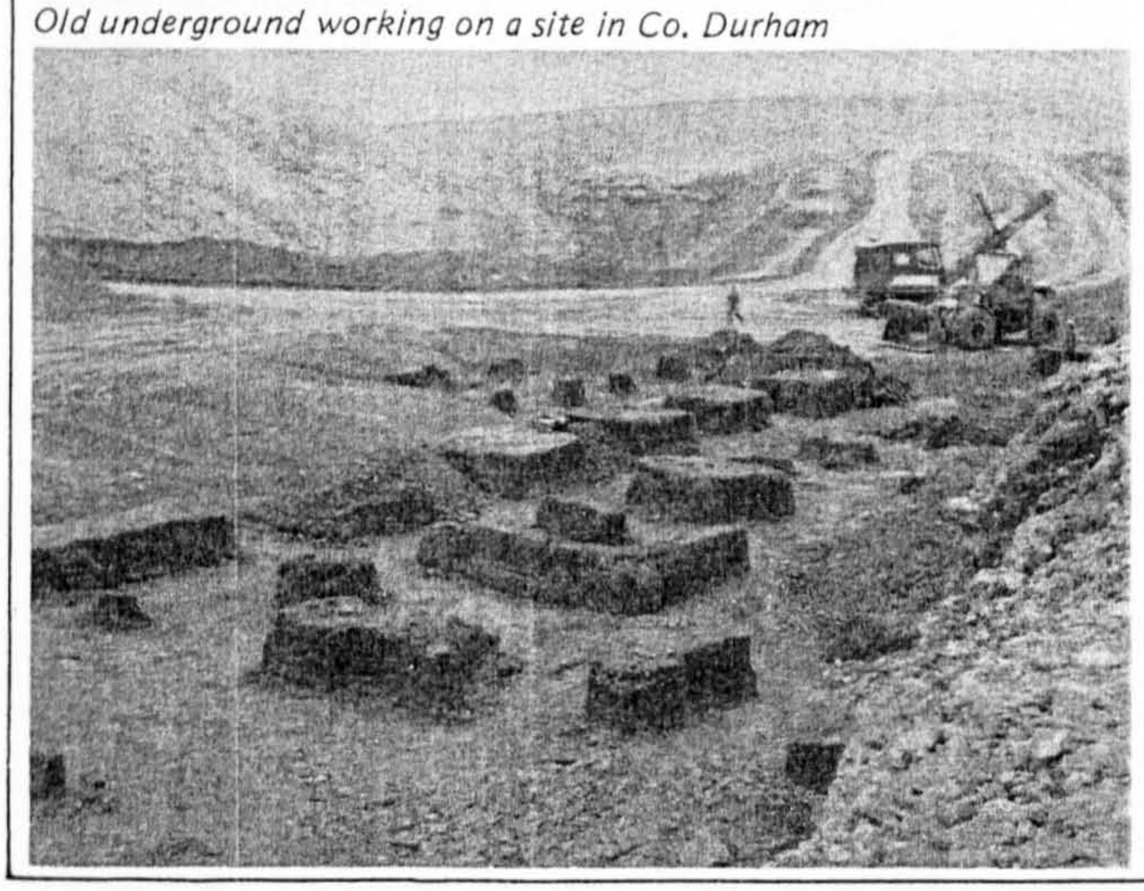
Failures in a soil mass usually involve shearing through the intact soil material, although fissuring, pre-existing shear planes, or other weak zones (eg lenses or bands of laminated clay) may have a significant effect. Also, soil slope failures often pass through the interface between the superficial deposits and the underlying bedrock.

Faulting and jointing

Faulting is probably the most significant feature involved in highwall and endwall failures. The faults themselves often consist of several weakness planes, or contain a zone of sheared or weaker material (gouge). There is an increased likelihood of closely spaced joints and steep digs in the faulted zones, with the probable existence of a joint set parallel to the fault plane(s).

Joint planes are much less continuous than either faults on bedding planes, but are closely spaced and usually two or more sets exist. Joints very frequently act as release surfaces for rock slope failures.

Old underground working on a site in Co. Durham



Slope failure due to presence of old workings

Bedding, seat earths and clay bands

Bedding planes form the most dominant system of discontinuities in a sedimentary rock mass, in that they are relatively closely spaced and continuous over relatively large areas. Also continuous over large areas in the Coal Measures are weak strata such as fireclays and clay bands (clay mylonites – intraformational shear zones[1, 2].

Old underground workings

Old underground workings are very frequently met in British opencast mining. These are mostly 'room-and-pillar' workings, but 'longwall' workings are also encountered. Due to collapse of the strata above the mined voids (void migration, longwall subsidence) bedding planes and joints tend to open up; also fracturing of the strata and localized steepening of bedding plane dip is caused.

Water pressures

Water pressure in the strata may reduce the stability of slopes by reducing the shear strength along potential failure surfaces due to uplift forces. Water pressure in tension cracks or similar near-vertical fissures reduces stability by increasing the forces tending to induce sliding.

Blasting

In order to facilitate excavation in the more competent strata (eg massive sandstones), it is often necessary to use blasting techniques to induce fragmentation within the rock mass. Blasting tends to loosen rock masses by discontinuity dilation, and this possible effect needs to be considered in critical situations. Also, the vibrations from the blasting signify the passage of dynamic forces through the ground, but these act only for a very short duration (milliseconds). The weights of charges used in opencast coal mining are relatively small compared with, say, blasting in hard rock quarrying for roadstone or aggregate production.

ROCK FAILURES

The types of rock slope failures which commonly occur in highwall and endwall situations are described below, and shown in fig. 3.

Plane failure

Plane failures are probably the most common type of rock slope failure encountered in British opencast coal mining. Sliding generally occurs along a plane or planes inclined towards, and daylighting within, the excavated face. The failure surface is usually a fault or faulted zone (fig. 3.1(b)), or a bedding plane containing a thin clay band or pre-existing shear zone (fig. 3.1 (c)).

Usually with a steeply inclined failure surface (eg fault) only one plane or narrow zone is involved, ie the failure is along a relatively straight surface from near the toe to behind the crest (fig. 3.1 (b)).

With a low-angle failure plane (usually bedding plane failures), tension cracks often occur, on joint planes or through superficials, near or just behind the slope crest, thus shortening the plane on which sliding occurs (fig. 3.1(c)).

Bedding plane failure involving sliding on weak clay bands has been known to occur where the dip of the strata has been as little as 1-in-18 (Stimpson and Walton[1]) although sliding on gradients less steep than 1-in-12 is fairly rare. In general, wherever the strata dip towards the excavation at a gradient steeper than 1-in-12, and the bedding planes intersect the excavation face, the possibility of failures should be investigated. Hydrostatic (groundwater) pressure and low shearing resistance along the weak horizon are usually the major contributory factors.

Sometimes sliding may also take place on an inclined secondary plane or planes, perhaps a fault, in which case the failure mode is usually referred to as biplanar or multiplanar (fig. 3.2(d) and (e)) (see Walton and Atkinson[4] and Stead and Scoble[5]).

Toppling failure

Toppling failures occur as a result of overturning rather than sliding. The usual case is where blocks are formed by a closely spaced and steeply inclined discontinuity system (most commonly jointing in British Coal Measures situations) dipping into the excavation face. The centre of gravity of each block must fall outside its outer lower corner for toppling to occur, which may then set the block falling freely. Hence toppling can be a particularly dangerous type of failure.

A common toppling failure situation is where the highwall advance towards and passes through a normal fault from the upthrow side (fig. 3.2(b)). A prominent jointing system sub-parallel with the fault creates 'overhanging' blocks which tend to topple while the excavation face is being advanced through this area.

Another common situation is where a normal fault is associated with bending of the strata or fault drag (fig. 3.2(c)). Here failure occurs as a combination of sliding forward of the blocks followed by toppling.

Wedge (tetrahedral) failure

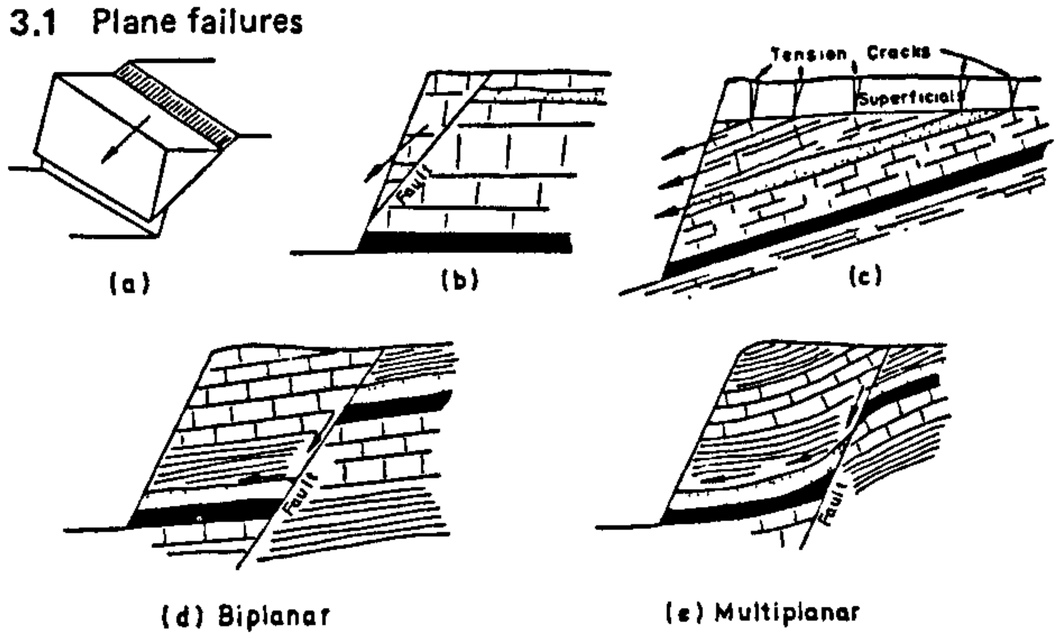
A tetrahedral wedge failure may result when two discontinuity planes intersect, and the line of intersection daylights within the excavation face. Sliding generally occurs along both the intersecting planes, which may be faults or joints or bedding planes. Wedge failures as observed in most British opencast mines are usually fairly small in size, but, because they often occur high above the base of the excavation, they can (like toppling failures) be very dangerous due to falling freely from the face.

Prandtl-type bearing capacity failure

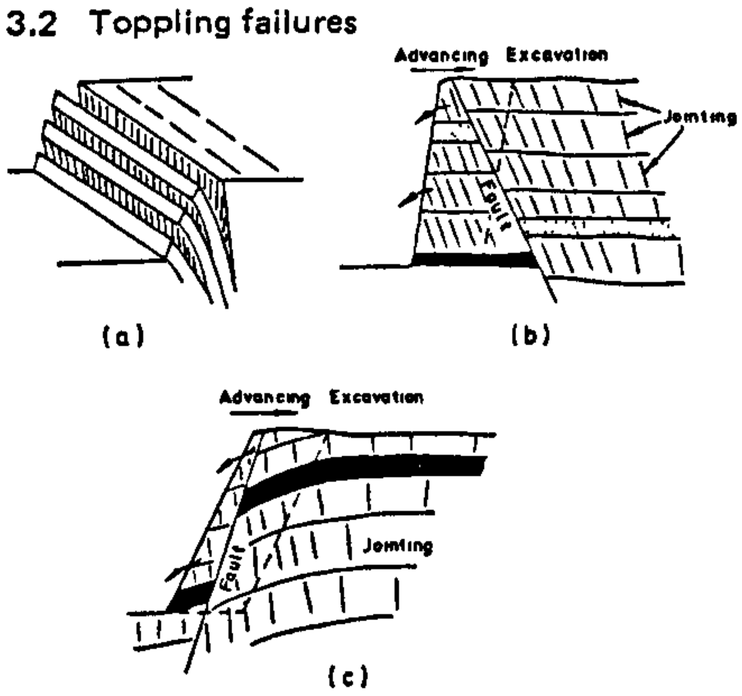
This mechanism has been proposed by Kvapil and Clews[6] to explain some of the very large slope failures which occur in opencast mining, and which quite obviously cannot be explained by the more simple mechanisms described previously. The main features are that, while considerable downward and outward movements take place at the crest of the slope, the outward movement at the toe of the slope is very much smaller. It is suggested that the movement at the crest is accommodated by upward and outward heaving in the middle part of the slope. This arises from the supposed action within the slope of active and passive wedges (see fig. 3.4). The name stems from the analogy to the classic bearing capacity failure mechanism originally proposed by Prandtl.

FIG. 3 Simplified rock slope failure models (based on Hoek and Bray[3], Walton and Atkinson[4], Stead and Scoble[5], Kvapil and Clews[6], Walton and Coates[7] and Walton and Taylor[8]

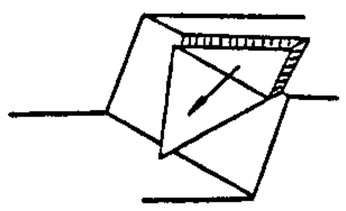
3.1 Plane failures



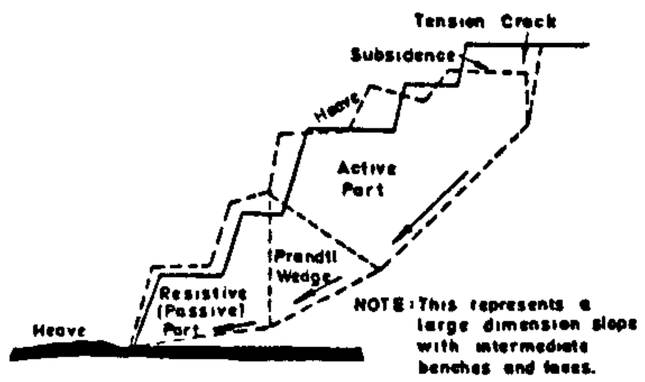
3.2 Toppling failures



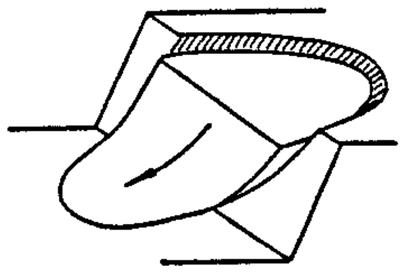
3.3 Wedge failure (tetrahedral)



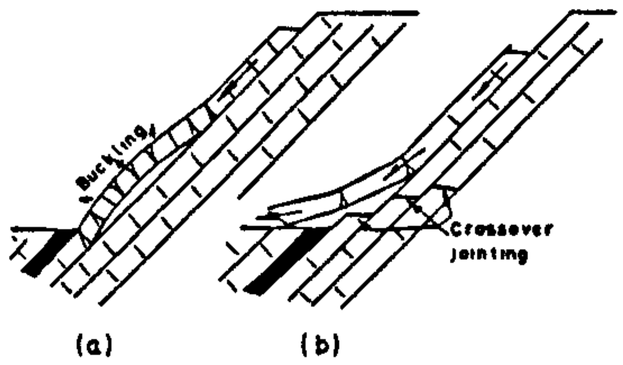
3.4 Prandtl-type bearing capacity failure



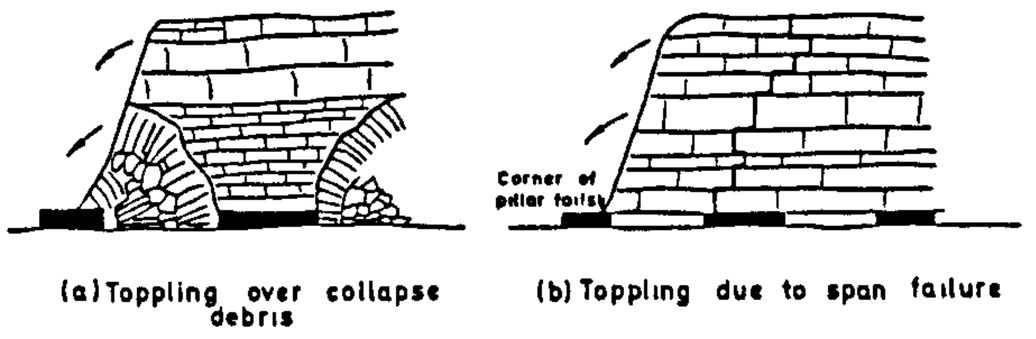
3.5 Circular failure



3.6 Footwall (slab) failures



3.7 Old workings failures



(a) Toppling over collapse debris

(b) Toppling due to span failure

Circular failure

Circular failure does not normally occur along pre-existing discontinuities (ie bedding, joint etc). Instead failure takes place mostly through the intact rock material. These failures usually only occur in very highly weathered rocks or clay quality rocks (eg marls) where the rock material is relatively weak and approaches the strength behaviour of an engineering soil. Characteristic signs of circular failure are arcing and stepped tension cracks behind the crest of the slope, and heaving at the toe.

Circular failures are, in fact, much more common in superficial materials and are discussed in more detail later.

Footwall (slab) failure

The situations illustrated are excavation faces in steeply dipping strata where the slopes have been formed at the angle of dip of the bedding. This exposes a slab or single bed of rock over the whole height of the face which, because of the presence of jointing or other weaknesses, buckels and/or slides downwards due to its own self weight (the buckling mechanism is an analogy with the buckling of brickwork). Hydrostatic pressure behind the face may also be a contributory factor.

This type of failure is generally associated with steeply dipping seams in south Wales (Walton and Coates[7]), Scotland, Co. Durham and west midlands, which necessitate the open-pit method of working. At Togston site in Northumberland the steeply inclined Acklington Dyke was exposed in an endwall excavation and the dyke itself failed by buckling.

Failures due to the presence of old workings

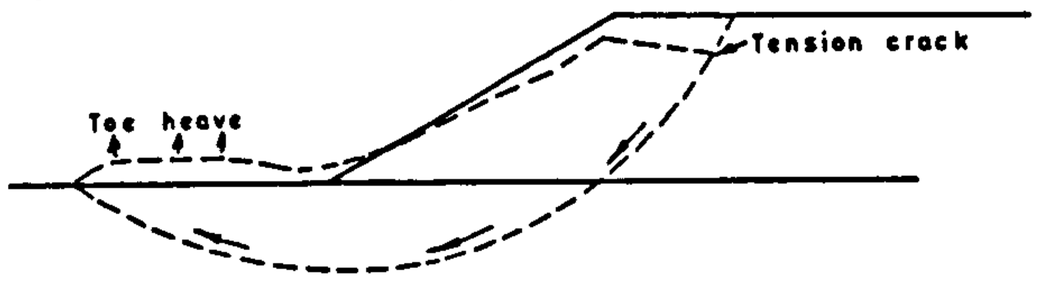
Wherever old workings (room-and-pillar or longwall type) are encountered below a high excavation face, slope failures may develop. The failure mechanism is very often of the toppling type (fig. 3.7(a) and (b)), although both plane and wedge-type failures also occur, dependent on the bedding and major joint orientations relative to the excavation face. Failures associated with the presence of room-and-pillar workings, whether uncollapsed or partially collapsed, have been observed to occur very rapidly and therefore can represent a serious safety risk. It is estimated that old workings are present in at least 25% of all highwall and endwall failures.

Papers by Walton and Atkinson[4] and Walton and Taylor[8] discuss the failure mechanisms in more detail.

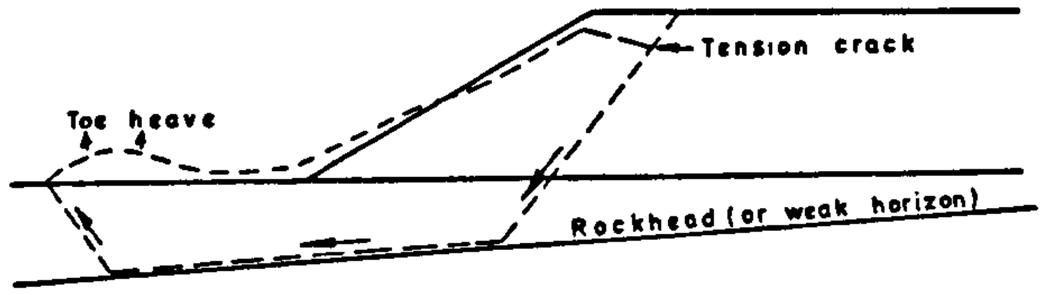
Rock falls

Rock falls are small-scale failures and occur when loose blocks develop on the face of an excavation. This is often due to the existence of closely spaced joints, which may have been opened up by the effects of weathering or blasting, or possibly due to stress-relief effects after excavation. Rock falls can be very dangerous because they may occur without warning and usually the blocks fall freely from the rock face.

4.1 Circular failure (Bishop[10])



4.2 Non-circular failure (or failure of general shape) (Janbu[11])



4.3 Biplanar failure (two wedge failure) (Sultan and Seed[12, 13] and Coulthard[14])

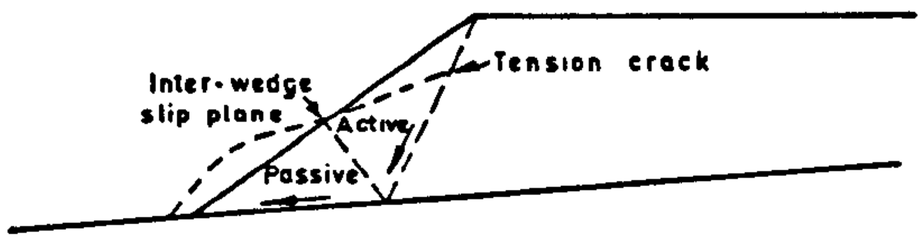


FIG. 4 Simplified soil slope failure models

Complex failures

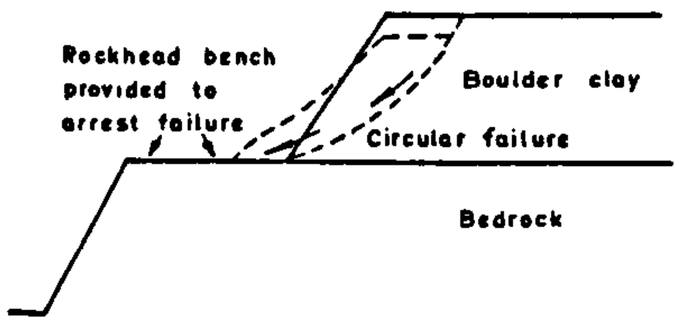
The descriptions and illustrations for rock slope failures given above represent a fairly simple classification. In practice, many slope failures seen at opencast sites will be easily recognized from the types described. However, some failures do not comply with simple form and may be complex forms being a combination of two, or even more, of the types given here. The larger-dimension failures particularly tend to be of more complex form and hence more difficult to recognize; for example, the failure at a site in Leicestershire described by Leigh *et al.*[9].

SOIL FAILURES

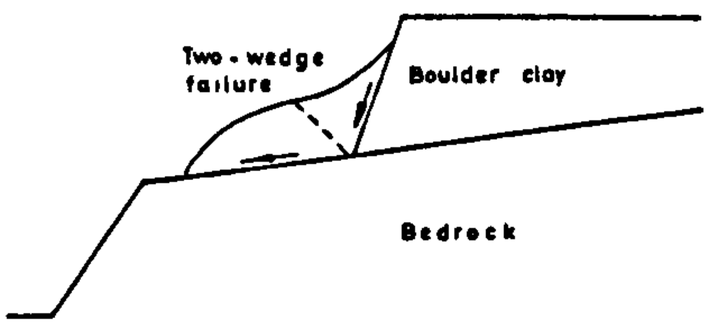
As stated earlier, slope failures in engineering soils are not usually defined by structural discontinuities, as in engineering rocks, but they may be influenced by the presence of a weaker soil stratum or the interface between superficial deposits and bedrock. The three most commonly recognized modes of failure are circular, non-circular and biplanar (also referred to as the active/passive or two-wedge mechanism) and are illustrated in fig. 4. Failures in engineering soils in the opencast mining environment can best be discussed by looking at where they occur, ie excavated slopes (being the upper parts of highwalls and endwalls), loosewalls and spoil mounds.

FIG. 5 Excavated slopes In soils (superficial deposits)

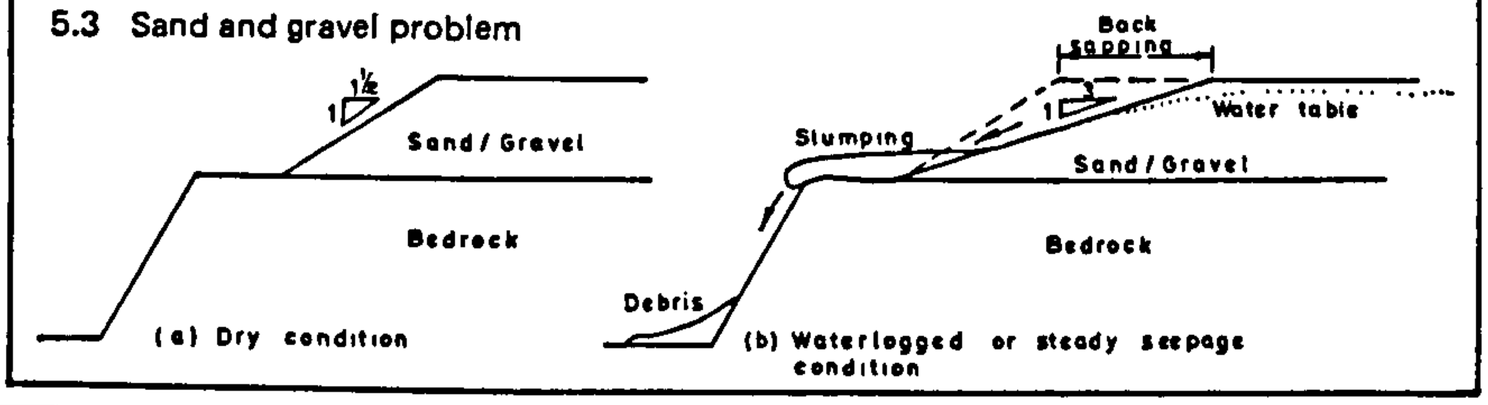
5.1 Oversteep slope



5.2 Steep rockhead gradient



5.3 Sand and gravel problem



Excavated slopes (highwalls and endwalls)

The circular type of failure on slopes excavated through soils is similar to that shown in fig. 3.5 for rock failures. Circular failures are probably the most common type witnessed in excavations in superficial materials, whereas they are not very common in rock slopes.

Figure 5.1 shows how circular failures occur above rockhead, especially if excavation is by crowd shovel; it also shows the importance of an adequate rock-head bench.

Figure 5.2 shows the effect of a steep rock-head gradient on the stability of the overlying drift deposits. Any situation where the rock-head gradient slopes towards the excavation at 1-in-12 or steeper is usually investigated to see if there is a potential stability problem.

Figure 5.3 shows the way in which sands and gravels behave when excavated. 'Dry' (above the water table) sand and/or gravel will stand at the 'dry' angle of repose, ie about 30–40°. If the granular deposit being excavated is waterlogged, however, then outward seepage forces (or buoyancy) reduce the stable slope gradient to about half of that for a 'dry' slope.

Loosewalls

Figure 6.1 shows the effect that a steep pavement has on loosewall stability. The tendency to slide depends on the strength and roughness, as well as the gradient, of the pavement rock. Seepage may also be a factor. The preferred course of action in this situation is to reorientate the direction of working so as to minimize the effect of the dip. An alternative remedial treatment is to create a rough surface by benching, ripping or blasting and so increase friction between the backfill material and the pavement.

Figure 6.2 shows how the loosewall face may be reshaped by the dragline to expose the coal edge and in so doing creates an oversteep face and unstable bench. Where loosewall material is rehandled, caution should be exercised to ensure that the angle of repose is not exceeded. This problem is concerned with the matching of cut width and dragline reach.

Spoil mounds (for the storage of overburden, subsoil or topsoil)

Figure 7.1 shows the deep-seated circular type of failure which commonly occurs when spoil mounds are placed over deep superficial deposits and built too high. The safe height of mounds can be increased by reducing the side slope gradients.

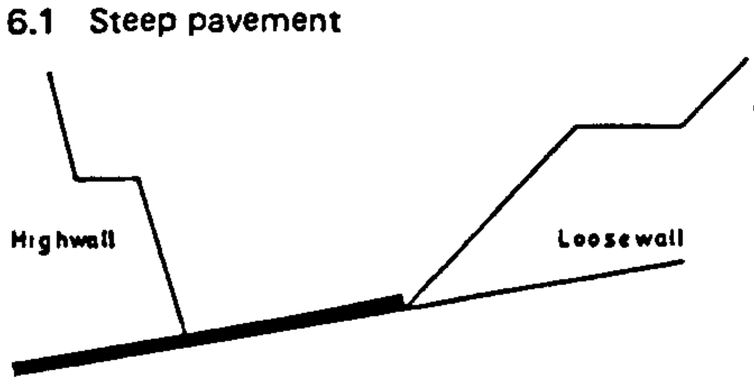
Figure 7.2 shows the two-wedge type of failure which may result when mounds are placed on steeply sloping ground. The solution to this problem is to excavate benches across the slope.

Figure 7.3 shows the situation where non-circular failures may occur, ie where the rock-head is not very deep, or where there is a relatively shallow weak stratum.

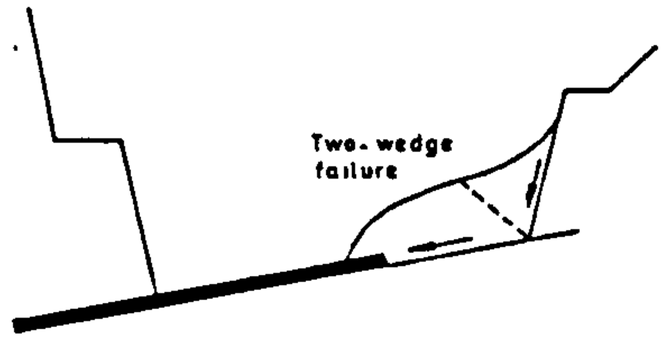
Figure 7.4 illustrates the stability problem which is created when superficial materials are placed in the lower layers of an overburden mound, eg where a site is overlain by a thick layer of glacial drift and this is excavated first to form the box cut. In this situation it is best to have a separate and lower mound for storing the drift material.

(Note that the types of failures shown for the various operational situations in figs 5, 6 and 7 are typical examples, and that it is feasible for different modes of failure from the ones illustrated to occur in the same situations.)

6.1 Steep pavement

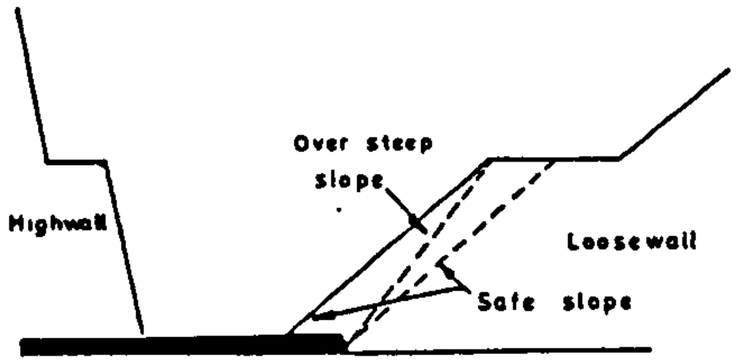


Critical gradient depends upon pavement rock type and seepage or groundwater forces
eg. Sandstone ≏ 1 in 8 to 1 in 9
Fireclay ≏ 1 in 10 to 1 in 12



Remedial treatment is to create rough pavement surface by benching ripping or blasting etc.

6.2 Face formed too steep



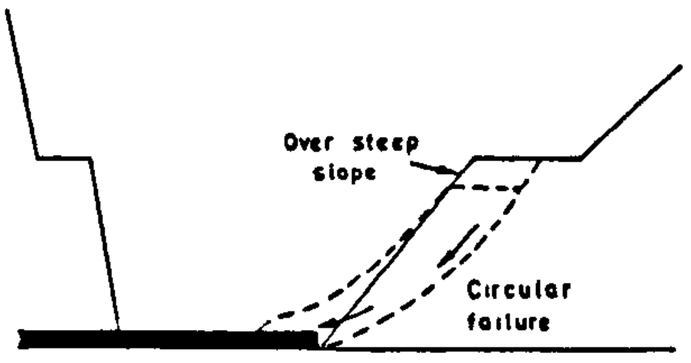
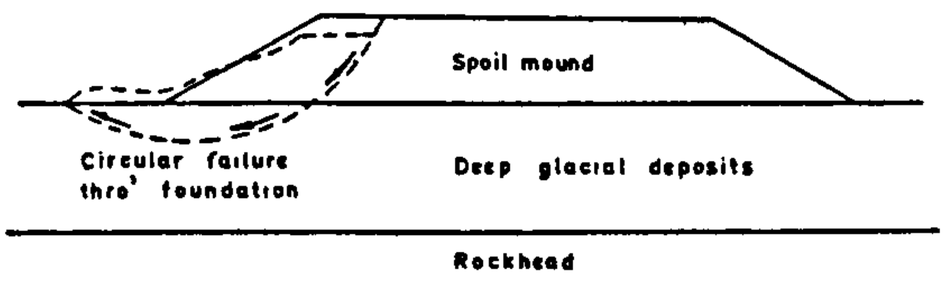


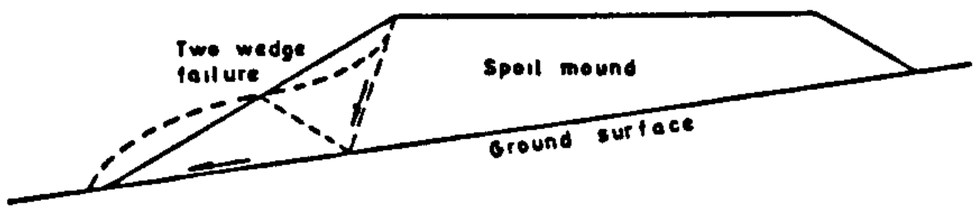
FIG. 6 Loosewall failures

Flow slides

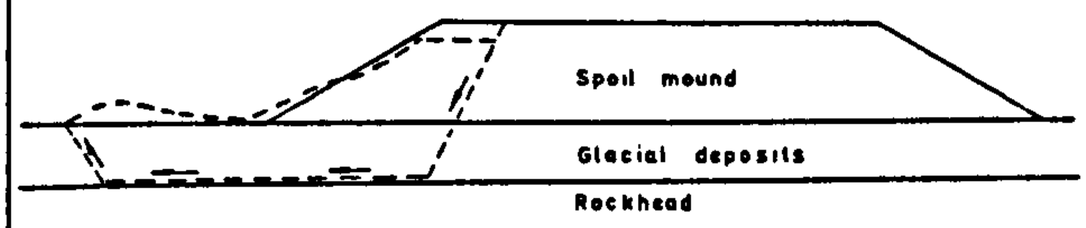
Liquefaction of spoil mounds or waste tips can occur when water pressure within the material rises to the point where the uplift forces exceed the weight of the spoil. This can occur if drainage systems are blocked or if the soil structure undergoes a sudden volume change (eg shock due to blast-



7.1 Mounds on deep superficial deposits

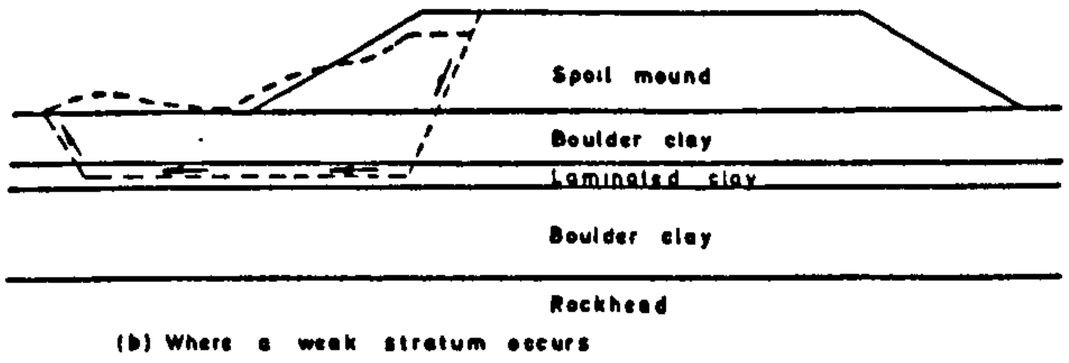


7.2 Mounds on sloping ground



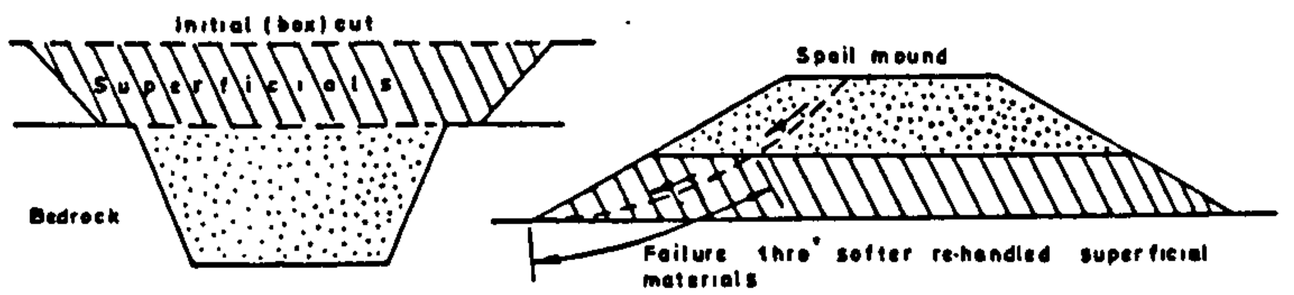
(a) Where rockhead is shallow

7.3 Non-circular (or general shape) failures



(b) Where a weak stratum occurs

FIG. 7 Spoil mounds



7.4 First layer of superficials problem

ing or earthquake). Liquefaction slides are very rapid and are therefore extremely dangerous. Fortunately such failures are fairly uncommon in Britain but the disaster at Aberfan in 1966[15] is generally accepted as having been caused by a liquefaction or flow slide.

REMEDIAL WORKS AND SAFETY MEASURES

Some remedial or preventive treatments have already been mentioned in previous sections of this paper, eg benching steep pavement slopes and steep ground slopes where either loosewall spoils are to be cast or spoil mounds are to be constructed, also the provision of rock-head benches in highwall and endwall slopes to arrest any failures which may occur in the superficial deposits.

The most commonly adopted remedial works and safety measures are given below.

Buttressing

Where the base or toe of a failure is near to the base of the excavated face or near to a sufficiently wide bench, then a buttress of excavated rock may be placed against the face and in front of the toe to provide extra passive resistance to further outward movement of the failing mass. Where a failure occurs in the higher part of a face, then a substantial volume of material may need to be placed beneath the unstable zone before the buttress construction reaches a sufficient height to start to have a restraining effect on the failure.

Benching

The provision of an intermediate safety bench or benches is often the most suitable way to contain small-scale failures (eg rock falls, wedge failures and toppling) as the free-fall distance is reduced from the full height of the excavation face to the height between benches. Where rock falls are a problem, regular inspections and scaling should be carried out.

With large instabilities the introduction of an intermediate safety bench or reduction of the slope gradient may reduce the size of the potential failure mass. Careful assessment is necessary, however, especially with planar failures and high water table regimes, as this can in some situations reduce stability doe to removal of 'dead weight' frictional resistance forces on the plane of sliding.

Dewatering and drainage

Dewatering may be necessary where high water levels are present within the rock mass. Pumping from deep wells or boreholes in advance of the excavation is sometimes used, as this keeps the water clear of the working area, improves slope stability and reduces pollution of pumped water discharge (Norton[16]). However, a pumping from sumps located at the lowest point of the working void is also a frequently adopted method.

The possibility of surface water run-off flowing into and filling tension cracks, thus exerting hydrostatic forces, must be guarded against and tension cracks sealed as they appear.

Slope monitoring

In some situations it may be required to monitor the ground behaviour adjacent to a deep excavation or high spoil mound. This may be because the possibility of a failure has been foreseen, or to give adequate safeguards to sensitive installations such as high-pressure gas mains. Simple surface surveying techniques are usually sufficient, but where there is evidence of the onset of an instability it is often prudent to install monitoring instruments within and adjacent to the potential failure mass. The types of instruments available include slip indicators, extensometers, inclinometers, tilt-stations etc. and it should be mentioned that the reading and processing of results from these instruments can be very time-consuming. The paper by Leigh *et al.*[9] describes how some of these methods were used to monitor a complex slope failure situation.

SLOPE STABILITY ANALYSIS

A detailed description of slope stability analysis techniques and design parameters is beyond the scope of this paper. These topics are discussed only briefly here, and the reader who wishes to pursue the subject in more detail is recommended to consult the reference indicated in the following sections and listed in full at the end of the paper.

Stability analysis techniques

The methods available for calculating the stability of slopes include limit equilibrium, probabilistic and finite element techniques. Of these, limit equilibrium methods, which compare disturbing and restoring forces or moments, are the methods in most common use.

In limit equilibrium methods the Factor of Safety (FoS) is defined as

$$\text{either } \frac{\text{Restoring forces}}{\text{Disturbing forces}} \text{ or } \frac{\text{Restoring moments}}{\text{Disturbing moments}}$$

and must be greater than unity to ensure stability. In fact, desired minimum FoS values usually range between 1.2 and 1.5 depending on situation and confidence in shear-strength parameters. The effect of forming a slope is to induce shear stresses within the adjacent ground mass which are resisted by the shear strength of the strata. In rock slopes a comparison is carried out in relation to existing discontinuities where these provide for 'kinematically possible' sliding masses, as shown in fig. 3. In soil slopes it is usually necessary to carry out a comparison for a number of possible or trial slip surfaces in order to obtain the minimum FoS.

The textbook by Hoek and Bray[3] is suggested as an introduction to limit equilibrium methods as it includes simple to use stability charts and graphical methods, as well as some worked examples. There are chapters on plane, wedge, circular and toppling failures. The methods of Bishop[10] for circular failure analysis and Janbu[11] for non-circular or general shape failures are also introduced in Hoek and Bray. It is quite possible to apply both the Bishop and Janbu analyses without resorting to computers, but this can be very time-consuming and several 'packages' for use with microcomputers are available commercially which will handle both methods.

An analysis for biplanar or two-wedge type failures is given by Sultan and Seed[12, 13] who apply it to sloping core earth dams. Also Coulthard[14] discusses the two-wedge method in relation to the back-analysis of spoil failures.

Shear strength parameters

The methods of drilling and boring used to penetrate the strata, and thus obtain samples, have been described earlier. In rock strata it is the shear strength parameters along the discontinuities (bedding planes, joints, faults etc.) which are important in slope stability analysis. Bedding planes and joints can be identified in rock cores, and Hoek and Bray[3] describe a portable shear-box apparatus which can be used for testing rock cores in the field. However, it is generally desirable to test rather larger discontinuity surfaces than is possible with cores, and for this purpose block samples are usually obtained from field exposures such as working opencast sites. The National Coal Board Opencast Executive has co-operated with the Mining Engineering Department of Nottingham University over a period of several years in carrying out research into the physical properties and shear-strength behaviour of Coal Measures rocks and the results of this work have been published by Hassani and Cassapi[17], Hassani and Scoble[18] and Denby *et al.*[19].

With regard to shear-strength parameters for superficial deposits, a site investigation involving cable percussion

boring and soil mechanics laboratory testing is now carried out prior to work commencing on every NCB production site[20]. The method usually adopted for determining the shear-strength parameters of superficial materials is the triaxial compression test which is included in BS 1377[21] and described in much detail by Bishop and Henkel[22].

CONTRACTUAL AND STATUTORY ASPECTS

In Britain opencast coal is mostly won from sites which have been 'authorized' by the Secretary of State for Energy, under the Opencast Coal Act 1958. From March 1984 the NCB Opencast Executive has to obtain planning permission for the sites, and in addition obtain authorization from the Secretary of State for the Environment; however, the working or production phase of the sites (ie the winning of the coal) is carried out by specialist civil engineering or opencast mining contractors under contract to the NCB. There are some items included in NCB Opencast Executive contracts which relate specifically to stability. Also there are several clauses about safety and stability included in both statutory and contractual documents. The following is only a brief summary.

Stability assessment

The object of the Stability Assessment is to describe the ground conditions at the proposed site in relation to slope stability and other geotechnical matters. The report is provided with tender documents and summarizes geological and geotechnical information gathered during the prospecting and feasibility investigation phases for the planned opencast site. It also discusses any factors which may affect safety or stability during the production phase of the site.

Code of Practice for Spoil Mounds[20]

This Code of Practice has been drawn up by a joint working party nominated by the Federation of Civil Engineering Contractors and the NCB Opencast Executive, and in consultation with HM Inspectorate of Mines and Quarries (Health and Safety Executive). The whole of the Code is given in a booklet pubished by FCEC[20] and is now applied to all new NCB opencast mining contracts. Basically, the Code requires that areas available for the construction of spoil mounds are defined and a site investigation of the superficial deposits in these areas is carried out. This information, which should include borehole data and soil mechanics laboratory test data, is given to all tenderers, being part of the contract documents. The successful tenderer, on being awarded the contract, is then required to design the proposed spoil mounds using soil mechanics principles, and to carry out regular inspections during and after construction.

Contract requirements

The NCB Opencast Executive form of contract for the operation of opencast coal sites includes several clauses relating to stability. These clauses draw attention to statutory regulations[23], the Stability Assessment and the geological information provided in the contract; also to the Code of Practice for Spoil Mounds[20]. Tenderers' submissions are required to refer to slope and benching geometry and to other stability matters.

Statutory requirements

The safety requirements for working opencast coal mines are set out in 'The law relating to safety and health in mines and quarries — Part 4; Quarries'[2, 3]. The following regulations relate to stability and some brief extracts from their contents are given:

Regulation 108(1) and (2) of the Mines and Quarries Act 1954 requires that the quarrying operations are so carried out as to avoid danger from falls. Also that in no part of the quarry shall the face of sides of the quarry be so worked as to cause overhanging, unless specifically exempted.

Regulation 2(1)(a) and (b) of the Quarries (General) Regulations 1956 requires that daily inspections shall be carried out of all working places, roads, top of quarry; also face, side or overburden of the quarry which may cause danger at any such place or road. These inspections to be reported on M&Q form no. 236.

Regulation 3(1) of the Quarries (General) Regulations 1956 requires that the overburden at or near the top of a face or side of the quarry shall be cleared back to a sufficient distance and depth to avoid danger from falls. Regulation 3(2) refers to exemptions from this requirement.

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PAPER C3 DEWATERING

Minett, S.T., Blythe, D.A., Hallam, G.D. and Hughes, D.B. 1985.

Analysis of an advanced dewatering scheme at an opencast coal site in Northumberland.

Groundwater in Engineering Geology Conference, Sheffield, 1985. Proceedings of 21st Annual Conference of the Engineering Group (eds. J.C. Cripps, F.G. Bell and M.G. Culshaw), Geological Society, London, 347-352.

"ANALYSIS OF AN ADVANCED DEWATERING SCHEME AT AN OPENCAST COAL SITE IN NORTHUMBERLAND"

S.T. Minett*, D.A. Blythe+, G.D. Hallam+ and D.B. Hughes+

ABSTRACT

A triangular area of Coal Measures rocks bounded to the south by the major WSW-ENE trending Causey Park Dyke, to the north and west by the limit of outcrop and the east by the North Sea, has been extensively worked by opencast methods over the past twenty years. Due to the close concentration of opencast workings it has been feasible to maintain a regional dewatering scheme over a number of years from Hauxley Shaft. However, during prospecting operations for an opencast coal site it was found that substantial quantities of groundwater existed in old workings horizons, ponded against the Causey Park Dyke in a broad anticlinal structural trap.

It was decided by the Contractor that the area should be dewatered before excavations took place, to avoid the danger of flooding and water induced slope failures, and a submersible borehole pump was, thus, installed.

This provided an opportunity to undertake a pumping test in mining disturbed strata. Due to the high permeability of the old underground workings it was found that the data did not conform to any standard analysis methods. However, it was found that the system could be modelled by treating it as a lagoon of triangular wedge shape. A mass balance calculation was performed and it was predicted that it would take approximately 350 days to dewater the site. The field data conforms closely to the predicted drawdowns and it is concluded that this method may be applicable to many regions with a mining history.

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INTRODUCTION

The most northerly section of the exposed coalfield in Norhumberland forms a triangular area bounded to the south by the major WSW-ENE trending quartz-dolerite Causey Park Dyke, to the north and west by the limit of outcrop and to the east by the North Sea. The area has been worked by deep coal mining methods for over 100 years, the last colliery closing in 1967.

Opencast operations have taken place in North Northumberland for more than 40 years. Initial high regional groundwater levels and the large size and depth of opencast sites meant that some form of dewatering was necessary. Due to the close concentration of opencast workings it has been feasible to maintain a regional dewatering scheme at Hauxley Shaft. The pumping scheme influences a very large area due to the high mass permeability of strata which are affected by old underground coal mining. This has maintained groundwater levels below the pavement seams on most sites at a level approximately 65 m B.O.D. However, during prospecting operations it was found that substantial quantities of groundwater existed in old workings horizons ponded against the Causey Park Dyke in a broad anticlinal trap.

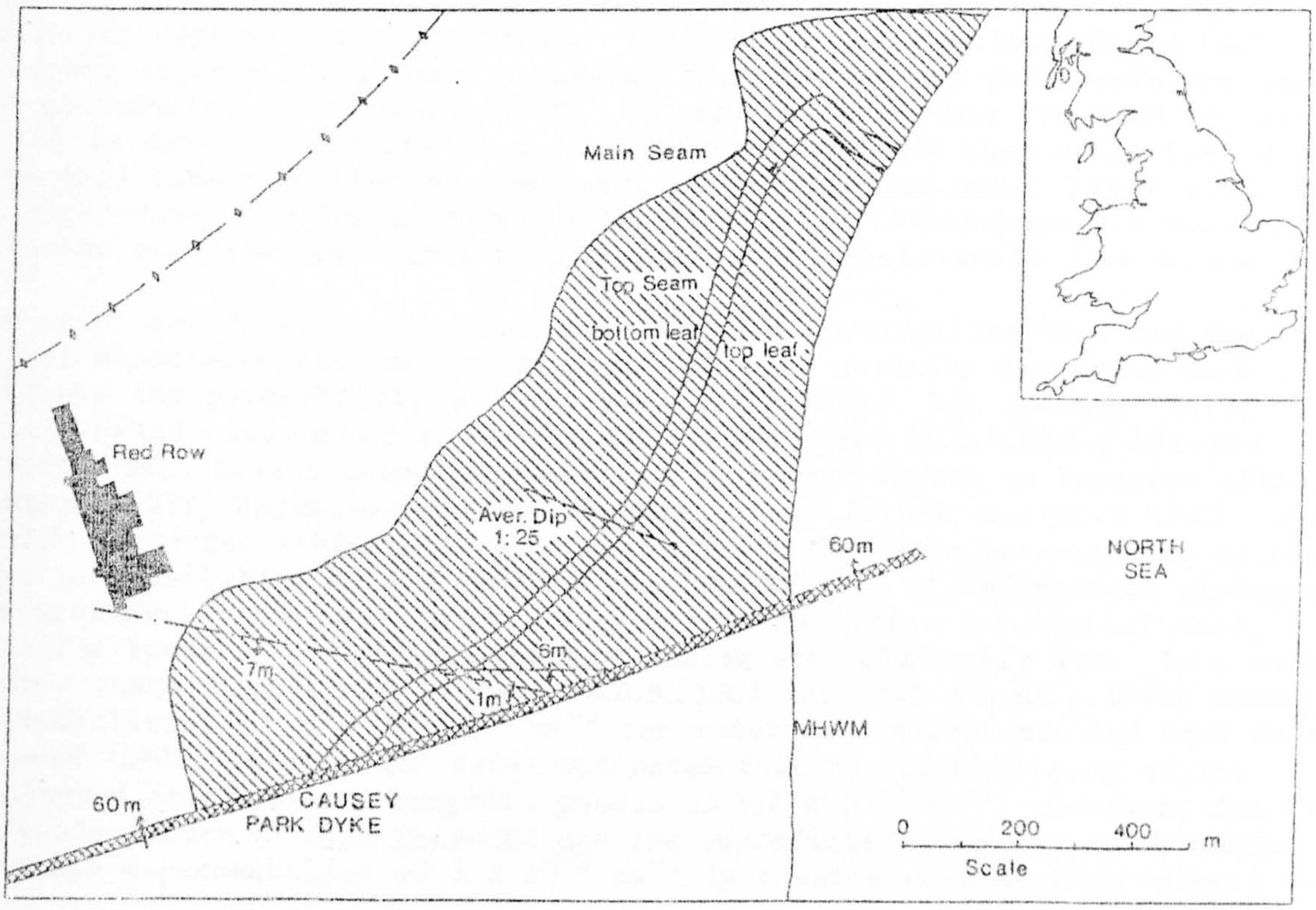

Figure 1 A location map of the area. The hatched area is the extent of the flooded old workings in the top and bottom leaves of the Top of Broomhill Seam and the Main of Broomhill Seam.

The geology of the site is simple. The strata form a typical Coal Measures sequence of interbedded sandstones, siltstones and mudstones with intercalated coal seams and seat earths. The sandstones, which comprise approximately 30% of the succession, are generally fine to medium grained, well cemented and

laterally impersistant. The strata dip gently to the south east at 2° being truncated to the south by the Causey Park Dyke. There is no major faulting on the site apart from a 60 m north throwing fault, along which the dyke is intruded. (See Figure 1)

The area is covered, as is usual in this region, by drift, which thickens to the south. It is dominated by clayey till with impersistant lenses of sand and gravel. To the south of the Causey Park Dyke there is a major W-E trending drift channel which is over 45 m deep and is mainly sandy in nature.

The hydrogeology of the area is controlled by four main factors:-

a) The pumping at Hauxley Shaft.

b) The proportion, distribution and type of old workings.

c) The Causey Park Dyke.

d) The drift cover.

Before opencast mining operations started on the site in 1983 the groundwater table was approximately 45 m B.O.D. Initial investigations (N.C.B.(O.E.) internal reports 1977, 1982) suggested that the Hauxley pump would not lower the groundwater level any further. It was concluded that provided the water level is maintained below 65 m B.O.D. at Hauxley Shaft then water levels on the site will remain unaffected. However, if the regional water level was allowed to rise above 65 m B.O.D. then groundwater would flow through old workings horizons over the crest of anticline and water levels would rise on the site.

The major aquifers of this system are the old workings horizons and the broken strata associated with mining activities. Some attempts have been made to estimate the permeability of Coal Measures strata. D.J. Hammond (1977, Unpublished data) and D.A. Blythe and K.G. Walters (N.C.B.(O.E.) internal report, 1982) have studied permeability in virgin strata on opencast sites, and Hammond (1977, Unpublished data); Sgambat et al (1980); Garritty (1980; 1982; 1983); McWhorter (1981) and Aston and Singh (1983) have attempted to estimate mass permeabilities in old workings and the effects of underground mining on the groundwater regime. In this region permeabilities for typical Coal Measures rocks in areas unaffected by mining are relatively low. In a suite of packer tests Blythe and Walters (N.C.B.(O.E.) internal report, 1982) found permeabilities of 10^{-6} to 10^{-7} ms^{-1} for mudstones, sandstones and coal seams. Hammond (1977, Unpublished data) estimated that the permeability of the collapsed strata above longwall panels is 5.7×10^{-5} ms^{-1}. However, due to the variable nature of roof measures and the unpredictability of caving above workings a permeability of 5×10^{-5} ms^{-1} is considered a minimum value.

The area of flooded old workings is terminated by the Causey Park Dyke. The strata to the south of the dyke are unaffected by mining and will consequently have lower permeabilities. Piezometer data indicate a standing water level of 25 m B.O.D., substantially higher than the northern side. It is assumed, therefore, that the dyke is a hydrogeological barrier with the pumping from Hauxley Shaft having little or no affect on the groundwater regime to the south.

The drift largely isolates the system from rapid infiltration from the surface

or from the sea. The water from the well has a Chloride concentration of 1000ppm indicating that sea water constitutes approximately 5% of total.

The dewatering well is 82 m deep (71 m B.O.D.) and is placed in an open old workings readway in the lowest worked seam. A 10"(250mm) Sumo submersible pump was installed which has a measured rate of discharge of $6.26 \times 10^{-2} m^3 s^{-1}$. Water from the well drains to the sea, via a lined ditch and a set of lagoons and, thus, does not return to the system.

ANALYSIS OF DATA

It has been found, while analysing this data, that the system does not conform to any standard aquifer testing methods, Cooper-Jacob analysis (Cooper and Jacob, 1946), Theis method (Theis, 1935), Theim method (Theim, 1906) and Stallman method (Ferris et al., 1962) for porous aquifers and the Kozeny-Carman equation and its derivatives (Kozeny, 1927; Carman, 1956; Sharp, 1970; Garritty, 1980) for fractured aquifers.

It is possible, however, to conduct a mass balance calculation, if a number of assumptions are made, to predict the rate of drawdown, time taken for total drawdown and recharge rate. These assumptions are

i) That the system was in equilibrium before pumping started (i.e. recharge to the system equalled discharge from the system).

ii) That the system can be approximated to á triangular wedge (see Fig. 2), the top surface (ABC) of which it the regional water level at 45 m B.O.D. and the over dimensions are shown on figure 2.

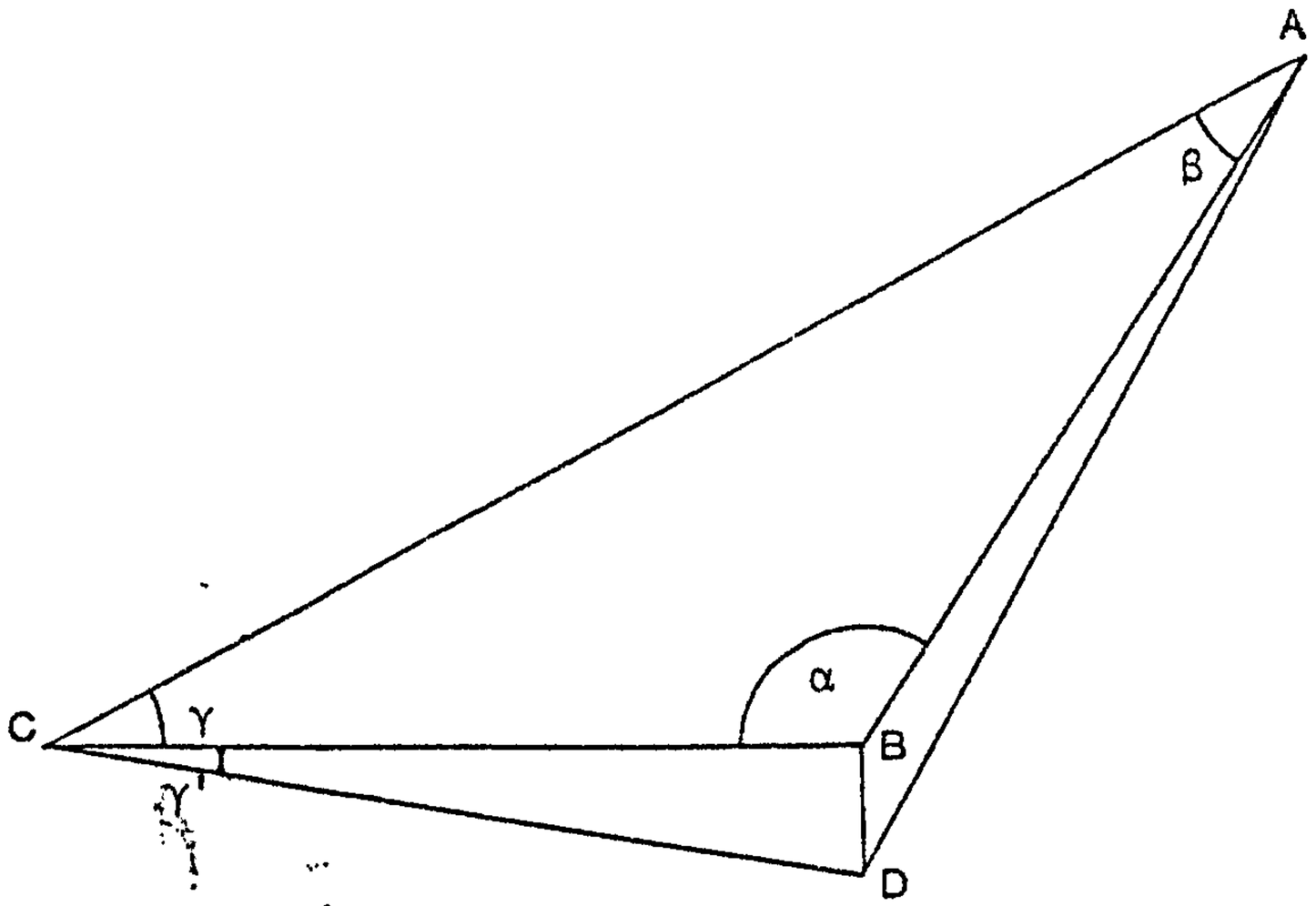

Figure 2 The approximate shape of the groundwater system, $\beta = \gamma = 30^{o}$; $\alpha = 120^{o}$; $\gamma' = 0.69^{o}$ and before dewatering starts CB = BA = 2150m; BD = h = 26m

iii) That all water emanates from the voids in the old workings. The minimum initial volume of water is 1.02×10^6 m^3 (Blythe, N.C.B.(O.E.) internal report, 1982). The maximum initial volume of water is equivalent to the total volume of coal removed, 3.81×10^6 m^3.

iv) That the flow of water into the well is instantaneous and that the upper surface (ABC) drops uniformly across it surface (i.e. that there is no cone of depression and that the groundwater reservoir can be regarded as a lagoon).

Thus $$P = 0.5 (h \cot \gamma')^2 \sin \alpha \quad (1)$$

and $$V = 1/3\ Ph \quad (2)$$

where P is the area (ABC) of the top surface and V is the volume (ABCD) of the wedge. As cot γ' and sin α are constants, then

$$V = 995\ h^3 \quad (3)$$

and $$dV/dh = 2985\ h^2 \quad (4)$$

so $$dV/dt = dV/dh \times dh/dt \quad (5)$$

where dV/dt is the effective pumping (discharge) rate and dh/dt is the drawdown rate.

If the assumption is made that throughout pumping the recharge to the system equals its natural discharge then

$$dV/dt = Qw^{-1} \quad (6)$$

where Q is the measured pumping rate from the well of 6.26×10^{-2} m^3s^{-1} and w is the water ratio, which is found from the volume of water (V_w) and the total volume

$$w = V_w V^{-1} \quad (7)$$

Thus the water ratio is between 5.8% and 21.8% (from assumption iii) depending on the estimate of initial water volume.

Then $dV/dt_{(max)} = 1.08\ m^3s^{-1}$ and $dV/dt_{(min)} = 2.87 \times 10^{-1}\ m^3s^{-1}$

From equation 5 the drawdown rate can be expressed

$$dh/dt = dV/dt.(dV/dh)^{-1} = 3.62 \times 10^{-4}h^{-2} \text{ for } dV/dt_{(max)} \quad (8)$$

and $$dh/dt = 9.61 \times 10^{-5}h^{-2} \text{ for } dV/dt_{(min)} \quad (9)$$

h (m)	d_1	t_1	d_2	t_2	d_3	t_3
25	5.80×10^{-7}	21	1.54×10^{-7}	78	3.95×10^{-7}	31
24	6.29×10^{-7}	40	1.67×10^{-7}	150	4.29×10^{-7}	59
23	6.85×10^{-7}	58	1.82×10^{-7}	217	4.67×10^{-7}	85
22	7.48×10^{-7}	74	1.99×10^{-7}	278	5.10×10^{-7}	108
21	8.21×10^{-7}	89	2.18×10^{-7}	333	5.60×10^{-7}	130
20	9.06×10^{-7}	102	2.40×10^{-7}	384	6.17×10^{-7}	150
19	1.00×10^{-6}	115	2.61×10^{-7}	430	6.84×10^{-7}	168
18	1.12×10^{-6}	125	2.97×10^{-7}	471	7.62×10^{-7}	184
17	1.25×10^{-6}	135	3.33×10^{-7}	501	8.54×10^{-7}	198
16	1.42×10^{-6}	144	3.75×10^{-7}	541	9.64×10^{-7}	211
15	1.61×10^{-6}	152	4.27×10^{-7}	569	1.10×10^{-6}	222
14	1.85×10^{-6}	159	4.90×10^{-7}	595	1.26×10^{-6}	232
13	2.14×10^{-6}	164	5.69×10^{-7}	617	1.46×10^{-6}	240
12	2.52×10^{-6}	169	6.67×10^{-7}	636	1.71×10^{-6}	248
11	2.99×10^{-6}	174	7.94×10^{-7}	651	2.04×10^{-6}	254
10	3.62×10^{-6}	177	9.61×10^{-7}	665	2.47×10^{-6}	259
9	4.47×10^{-6}	179	1.19×10^{-6}	676	3.05×10^{-6}	263
8	5.66×10^{-6}	182	1.50×10^{-6}	684	3.86×10^{-6}	267
7	7.39×10^{-6}	184	1.96×10^{-6}	691	5.04×10^{-6}	269
6	1.01×10^{-5}	185	2.67×10^{-6}	696	6.86×10^{-6}	271
5	1.45×10^{-5}	186	3.84×10^{-6}	700	9.88×10^{-6}	273
4	2.26×10^{-5}	186	6.03×10^{-6}	702	1.54×10^{-5}	274
3	4.03×10^{-5}	188	1.07×10^{-5}	704	2.74×10^{-5}	274
2	9.06×10^{-5}	188	2.40×10^{-5}	705	6.17×10^{-5}	275
1.	3.62×10^{-4}	188	9.61×10^{-5}	705	2.47×10^{-4}	275
0	-	188	-	705	-	275

Table 1 Showing the drawdown rates, d_1; d_2; d_3 for $dV/dt_{(max)}$ $dV/dt_{(min)}$; $dV/dt_{(ob)}$ in ms^{-1} respectively and the corresponding drawdown times, t_1; t_2; t_3.

The rates of drawdown are presented in Table 1 and it is possible to calculate the time taken for total drawdown from equation 5

$$dh/dt = dV/dt \ (dV/dh)^{-1}$$

$$dt = 2985(dV/dt)^{-1}h^2\ dh$$

As the effective pumping rate is constant by setting $2985(dV/dt)^{-1} = k$ then:

$$dt = kh^2\ dh \qquad (10)$$

$$t = kh^3/3 + C \qquad (11)$$

where t is the time taken for drawdown to values of h and C is the integration constant.

This equation may be rewritten as:-

$$t = (9.95h^3 - 1.75 \times 10^7)/(dV/dt) \qquad (12)$$

It is, then, possible to draw a set of type curves for various values of dV/dt (Figure 3) and the limits, $dV/dt_{(max)}$ and $dV/dt_{(min)}$, may be plotted, the corresponding times for total drawdown are approximately 200 days and 700 days.

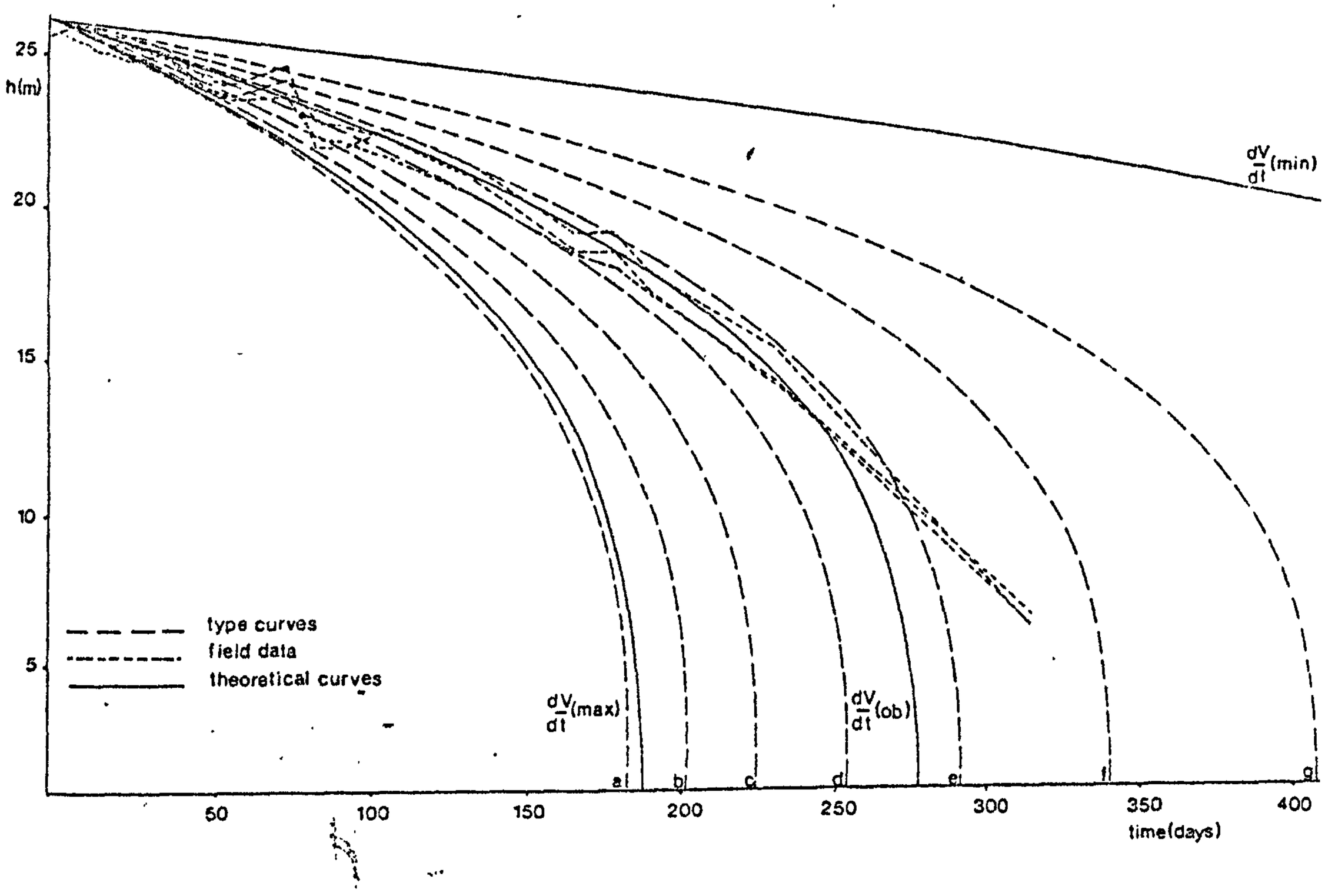

Figure 3 The type curves for drawdown against time for a range of dV/dt values, a=1.1 ms^{-1}; b=1.0 ms^{-1}; c=9.0x10^{-1} ms^{-1}; d=8.0x10^{-1} ms^{-1}; e=7.0x10^{-1} ms^{-1}; f=6.0x10^{-1} ms^{-1}; g= 5.0x10^{-1} ms^{-1}. Also plotted are the curves for $dV/dt_{(max)}$; $dV/dt_{(min)}$, $dV/dt_{(ob)}$ and the field data.

Using data from the field it is possible to predict an improved drawdown rate (see Figure 3 and Table 1) and the estimated time taken to dewater the system. An estimate has been taken on the first 108 days of the test or the initial 4m drawdown. The observed effective pumping rate ($dV/dt_{(ob)}$) is 7.37×10^{-1} m^3s^{-1} and the time for total drawdown is 275 days. It is also possible to calculate a water ratio and an initial water volume from the system based on observed data:

from equation 6 $w = Q(dV/dt_{(ob)})^{-1} = 8.5\%$

and

from equation 7 $v_w = w.V = 1.47 \times 10^6$ m^3

However, it is unlikely that the total recharge will equal the natural discharge once pumping starts. As the system is dewatered drainage from thesurrounding strata and leakage from the strata in the system will occur. This effective recharge will increase as the system is further dewatered. It was found that effective recharge started to have a significant effect on the predicted drawdown curve at 14m (a drawdown of 12 m) and the proportion of recharge increased at a steady rate of 1.6×10^{-2} m^3s^{-1} per metre as drawdown proceded. Effective recharge to the system can be seen in Figure 3 as the deviation of the field data plot from the calculated drawdown curve. Thus, it was estimated that the flooded old workings would take 375 days to dewater. This situation, however, was not reached as water from a surface pond in the opencast site was pumped into the system after 320 days, thus prolonging the dewatering.

It has been shown that in an area where a number of coal seams have been worked by underground mining methods the resulting highly fractured aquifer behaves as a pseudo-lagoon and limits may be put on the initial volume of water in the system, the drawdown rate and effective recharge to the system.

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PAPER C4 NORTHERN BACKFILL 1

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Symposium on Surface Mine Hydrology, Sedimentology and Reclamation, University of Kentucky, December 1985.

AN INVESTIGATION INTO GROUNDWATER RECOVERY AND BACKFILL CONSOLIDATION IN BRITISH SURFACE COAL MINES.

by

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ABSTRACT

The paper presents the instrumentation and results from an investigation into groundwater recovery and backfill consolidation conducted in the United Kingdom. British surface mining often requires the removal of roads or other structures which must be replaced on the cessation of backfilling. It is imperative for sucoessful reconstruction that degrees of surface settlement be evaluated and thus precautions against structural damage may taken. Methods of working and geological characteristics of the backfills investigated are also detailed. An outline of research being carried out with respect to groundwater pollution from mine backfills is presented.

Introduction.

Groundwater recovery within an opencast mine backfill is the process by which the natural water table in a mine out re-attains its equilibrium level following the cessation of pumping in a surface or underground excavation. The rate of re-establishment is governed by many factors including the depth of the depressed level, the permeability of the fill and the degree of recharge. Recovery in this way may occur over a wide range of time scales, days, months or even years.

There are two principal effects of groundwater recovery;

a). The increase in settlement rates of unconsolidated fill materials owing to chemical and physical effects of contact with groundwater flow, (the subject of this paper), and,
b). The pollution of groundwater flow owing to contact with certain contaminating materials within the spoil. This subject is discussed briefly at the end of the paper.

Both of these problems are being investigated by Nottingham University Department of Mining Engineering.

Opencast Coal Mining in the United Kingdom.

The mining of near surface coal deposits in the United Kingdom is typified by small scale operations. The total coal output by surface methods amounts to 14-15 million tonnes per year from an average of 55 working sites. This output constitutes around 12% of the total coal output of the United Kingdom. Sites vary in total production from 0.06 to 12.8 million tonnes and in area from 9 to 826 hectares. Depths of mining commonly attain 80 metres and exceptionally reach 200 m. Overburden ratios can be in the range of 20-30:1. Reclamation is a very important part of the surface mining operation and is strictly supervised. Topsoils and subsoils are stripped and stored separately during the coaling operations andare replaced directly following the backfilling. Land intended for agricultural use is managed for five years by the Ministry of Agriculture. Water supples, fencing, hedges and permanent drainage are all re-installed.

Investigations into the Settlement of Opencast Mine Backfills.

The working of sites with high overburden ratios (20-30:1), results in large areas of relatively deep unconsolidated backfill materials. This backfilled mass has the capability once emplaced, of under going significant settlements. In the United Kingdom much thought has been given to the use of backfilled sites for light industrial development and thus the degree and timescales of such movements have become of interest. Some structures have been previously erected upon backfilled sites, and occasionally have undergone severe structural damage owing to

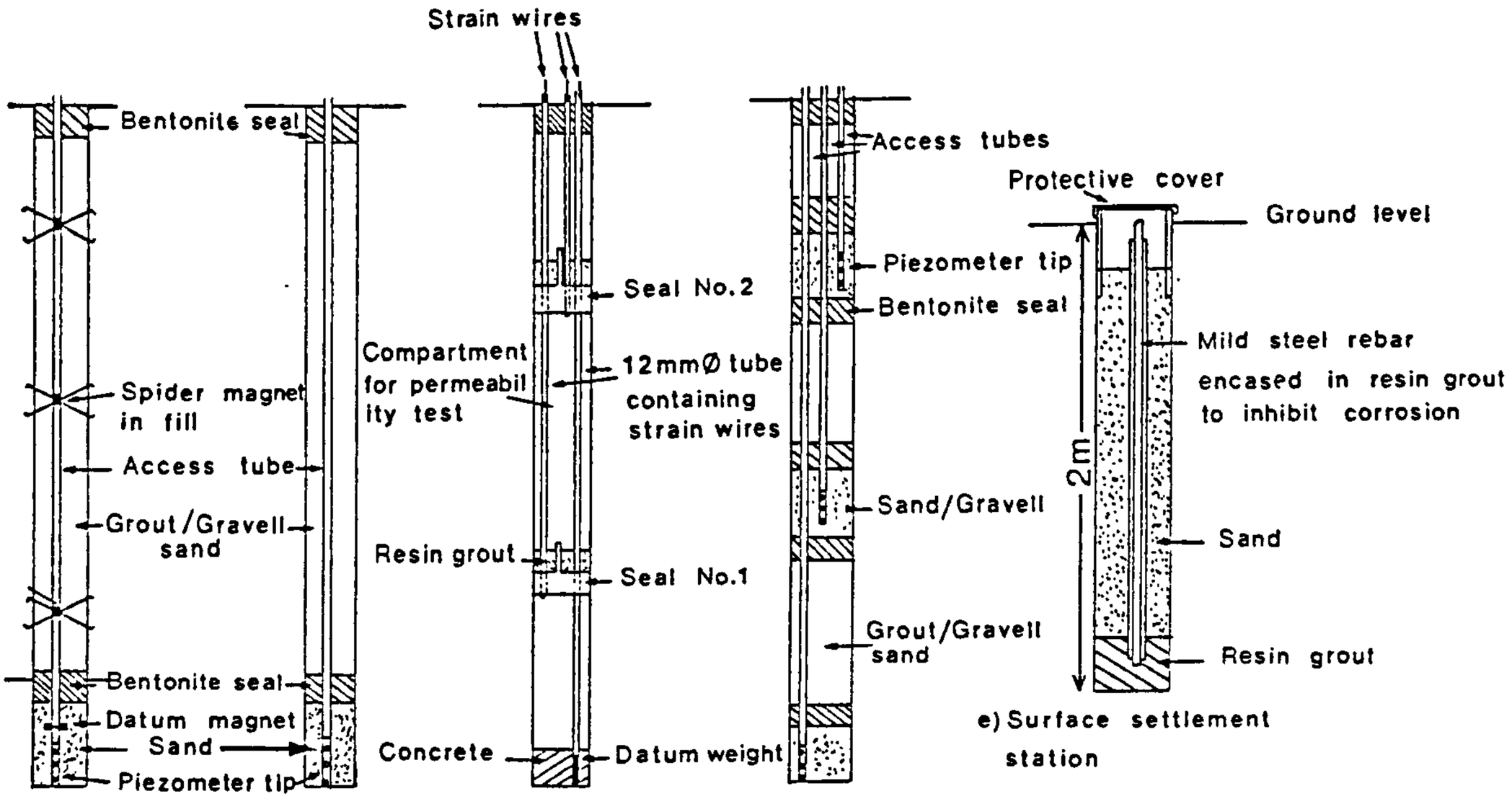

Fig.1 Instrumentation for backfill settlement projects

delayed groundwater recovery within the fill, Penman and Godwin (1974), Smyth-Osbourne and Mizon (1984). Research has shown that the principal cause of increased and accelerated backfill settlements has been a result of a recovering groundwater table, Charles, Hughes and Burford (1983).

The field investigations presented in this paper are currently being conducted on five opencast sites in the United Kingdom. The sites along with their instrumentation schemes are detailed in Table 1.

Current Findings.

Site A.

Site A was instrumentated in October 1984 and the full sequence of groundwater recovery was monitored within four months from the onset of monitoring. Backfill settlements recorded on the individual instruments has varied from a maximum of 0.25m (in 23m of fill), to only 0.031m, (in 19m of fill). The last result is of especial significance as this instrument was installed in the line of a former haul road which had undergone regular compactive loads by moving plant.

Groundwater levels at the outset of monitoring were standing at 11 m below the restored surface and had re-established to flood the surface within four months. The standing water was pumped off site during March 1985, and the restoration programme continued with the replacement of subsoil and topsoils. During this period the instruments were buried so as to protect them from plant damage. Recovery of the instruments is expected in August 1985.

At any one time the results on all the extensometer instruments showed there to be only one active zone of consolidation within the backfill mass, this corresponding to the surface of the recoverying water table. An example of the type of results obtained is given in figure 2.

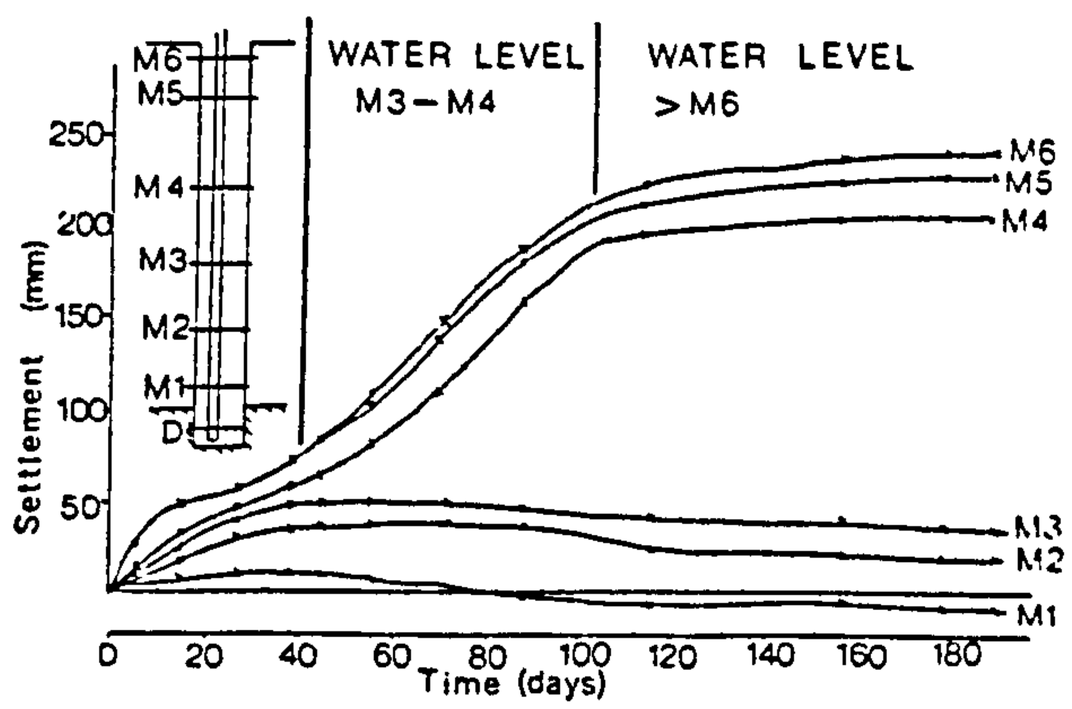

Fig 2. Site A - Extensometer Results

Table 1. Instrumentation Schemes for Backfill Settlement Monitoring

ME/P - Combined Magnet Extensometer/Piezometer
P - Standpipe Piezometer.
TWE - Tension Wire Extensometer with Permeability Testing Compartments.
MMP - Multi-Point Piezometer.
S - Surface Settlement Station.

Site and Instruments	Date of Commencement of Monitoring.	General Site Details.
A 5 x ME/P 4 x P	October 1984.	Backfilled Site lying in a flood plain. High groundwater levels. Shallow, (ave 17m), mudstone fill. Max. depth 30m. Instrument depths at 20m. Truck-Shovel-Scraper Operation.
B 2 x TWE 2 x MMP 4 x P	July 1985.	A dragline Site working to depths of 80m. Instruments in 75m of fill. Backfill predominantly mudstones and sandstones. water levels controlled by pumping in local deep mine shaft.
C 4 x ME/P	July 1985.	Site Pavement consists of synclinal basins with fill depths of 30 to 70m. Instruments have been installed along the line of a proposed road. Truck and Shovel Operation. Sandstone/Mudstone.
D 3 x ME/P 30 x S	February 1982.	Instrumentation to monitor the settlement of a sewerage pipe installed in the restored fill. Fill depths over monitored area, 15-30m. Dragline Site, Mudstone and Sandstone fill.
E 25 x S 3 Traverses.	May 1974.	Instruments positioned over 15-30m of dragline fill. Three traverses over the site. Initially instrumentation to monitor surface settlement alone, later tied into groundwater details.

Individual Instruments are illustrated in fig 1.

Site B.

Monitoring began in July 1985. A failure in a magnetic extensometer instrument assumed due to shearing movements within the fill led to the installation of two tension wire extensometer instruments, capable of monitoring shear as well as consolidation. These instruments have been modified to conduct permeability tests in a number of compartments within the borehole. Multi-Point Piezometer instruments have also been introduced into the programme to measure pressure differences within the fill horizons. Piezometers in the solid excavation surrounding the excavation monitor the recharge rates. At the date of this paper only initial readings have been obtained.

Site C.

Monitoring on Site C consists of the measuring of the deformation of a road constructed over a backfilled area. The road traverses several synclinal basins and the depths of fill under it can vary from 30 to 70 m over horizontal distances of 50 m. Installation was completed during June 1985 and the initial readings followed directly. Groundwater levels are expected to recover to surface giving a measurable recovery on the site in excess of 20 m. Substantial differential settlements are expected owing to the synclinal nature of the pavement strata. A section across the site showing instrumentation is shown in fig 3.

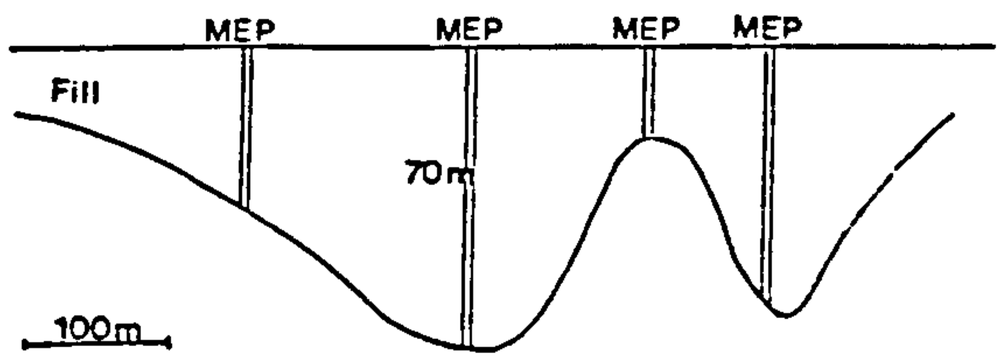

Fig 3. Site C - Instrumentation Scheme.

Site D.

The purpose of the Site D instrumentation was to monitor the deformation of backfill around a buried sewerage pipeline. Instrumentation was completed in February of 1982 and readings are still continuing. Figure 4 details a plan of the instrumentation whilst figure 5 details a section across the backfill which the pipeline traverses.

The sewer consists of 315 mm steel pipe, fitted with sliding joints at either end to permit elongation on a vertical curve.

The nature of the backfill consists chiefly of coal measures spoil with some boulder clay. Large blocks of sandstone are present several metres in thickness.

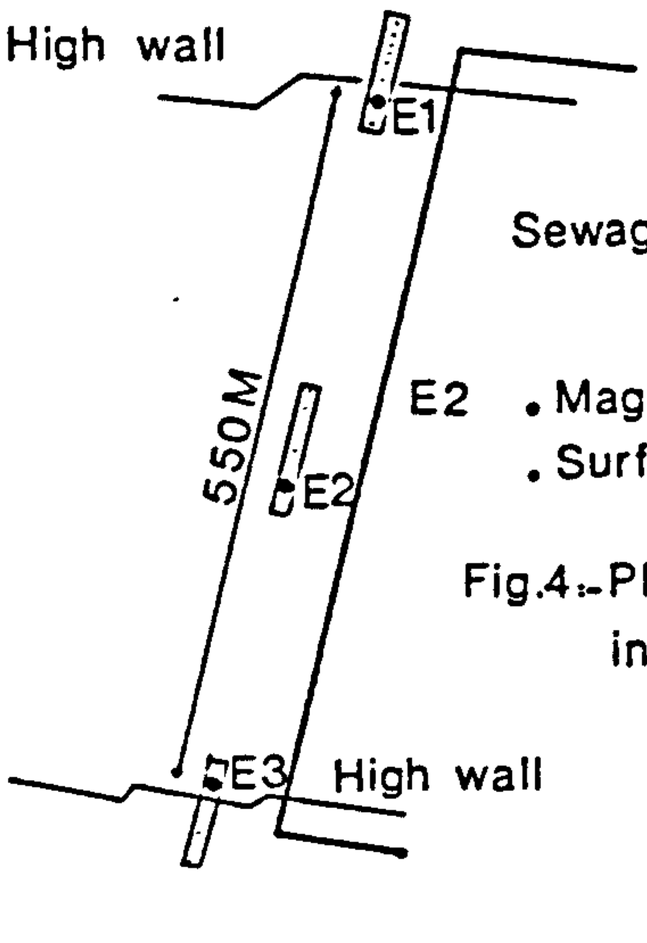

Fig.4.-Plan of site D instrumentation

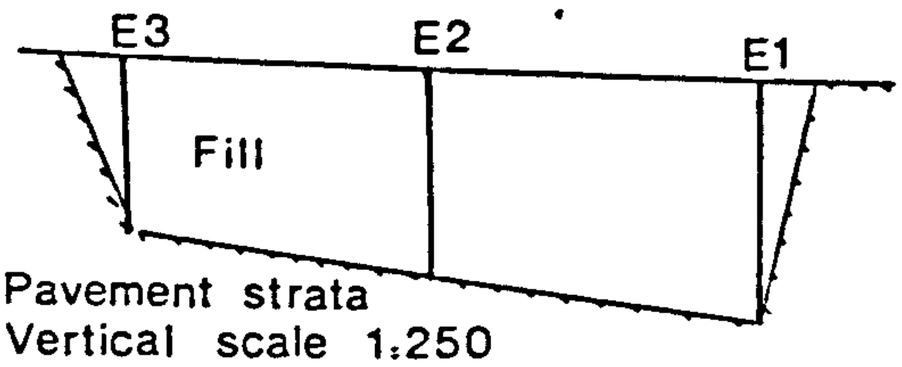

Fig.5.- Section of site D instrumentation

The instrumentation scheme consisted of the installation of three magnetic extensometers/ piezometers in the fill along with several surface settlement gauges complementing each. Extensometers were installed to depths of 16, 27 and 29 metres depth. The extensometers consisted of magnetic targets over a central access tubing. The movement of the targets reflected the displacement of the fill and could be recorded using a reed switch probe. Water levels are determined using a standard dip meter in the same access tubing. A datum magnet lies below the fill in the bedrock. Readings were taken at intervals of once a month for the first six months and then every two months for the next 18 months of monitoring and now continue at half-yearly intervals. Reading months are currently March and August so as not to interfere with local agricultural practise.

Settlement figures obtained from the monitoring have been very low and reflect the fact that no water recovery has occured in extensometers E1 and E2. On E1 a total surface settlement of 15 mm has been measured by precise levelling techniques. However on E2 and E3 heaving movements of 3 and 6 mm respectively have been registered. The fill has effectively not moved during the three years of monitoring. The degree of settlement of E1 was in fact the maximum displacement monitored on any instrument.

Groundwater levels for extensometer E3 are illustrated in figure 6, which indicates that during the monitoring period groundwater has fluctuated from the base of the fill to the surface. Settlement results for the individual targets on extensometer E3 are also given in figure 7.

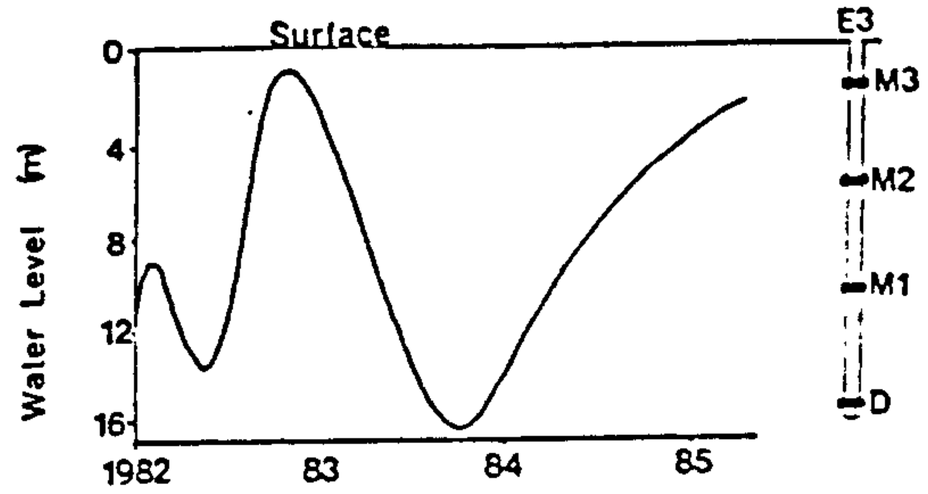

Fig 6 Site D. E3 Water Levels

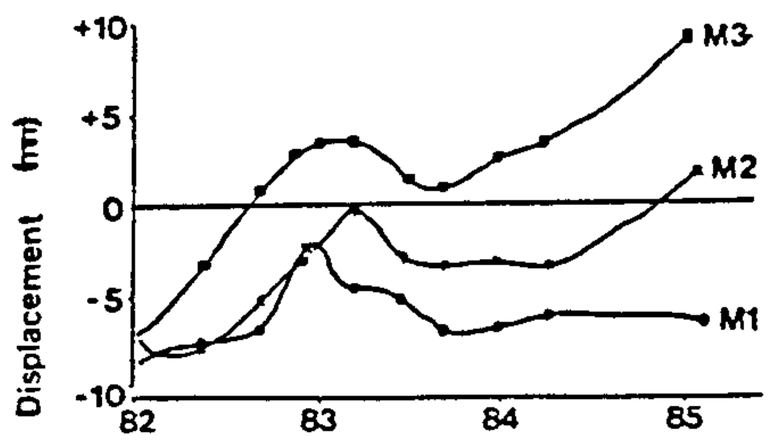

Fig 7. E3 Settlement Records 1982-5

It is suggested that adjacent opencast mine workings are partly responsible for the low groundwater levels, although the fluctuations of water in E3 are difficult to interpret. A complete analysis of the local groundwater recharge zones is required before further conclusions may be drawn. A piezometer malfunction cannot be discounted from this analysis.

The results from this site however do display the importance of groundwater recovery in determining the degree and rate of backfill settlement, and clearly show in cases of zero fill saturation settlements may possibly minimised to negligible quantities.

Site E.

Monitoring on site E commenced with surface levelling investigations in 1974. At the time of installation no emphasis had been applied to the presence of groundwater in the fill, and part of this research has been to interpolate groundwater levels from adjacent piezometer data to the locations of the surface settlement locations. Site E was a dragline site with a mudstone/ sandstone backfill in the same locality as site D.

Instrumentation on Site E comprises of three sets of surface levelling stations, (figure 8). Hydrologically and geologically the site is isolated from local opencast sites by the presence of an igneous dyke which prevents water flow across its line. Two piezometers are also isolated by the dyke and thus may be used to approximate groundwater levels in the backfill for this particular area.

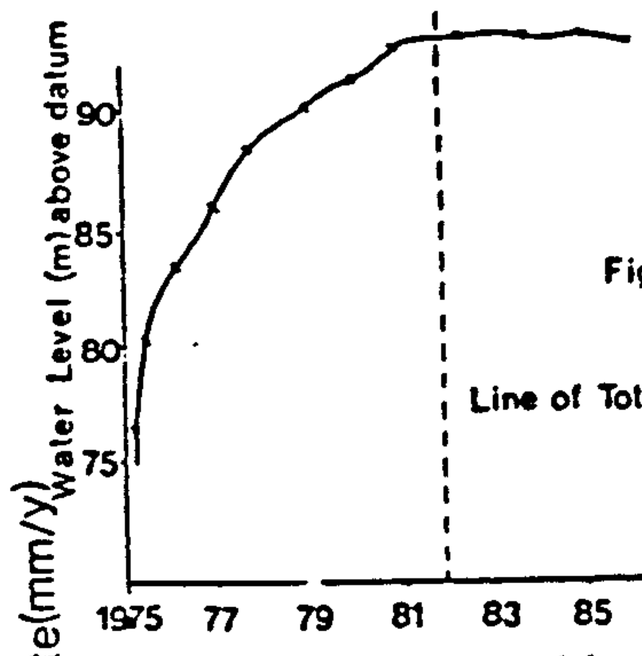

Fig 8 Water Recovery on Site E

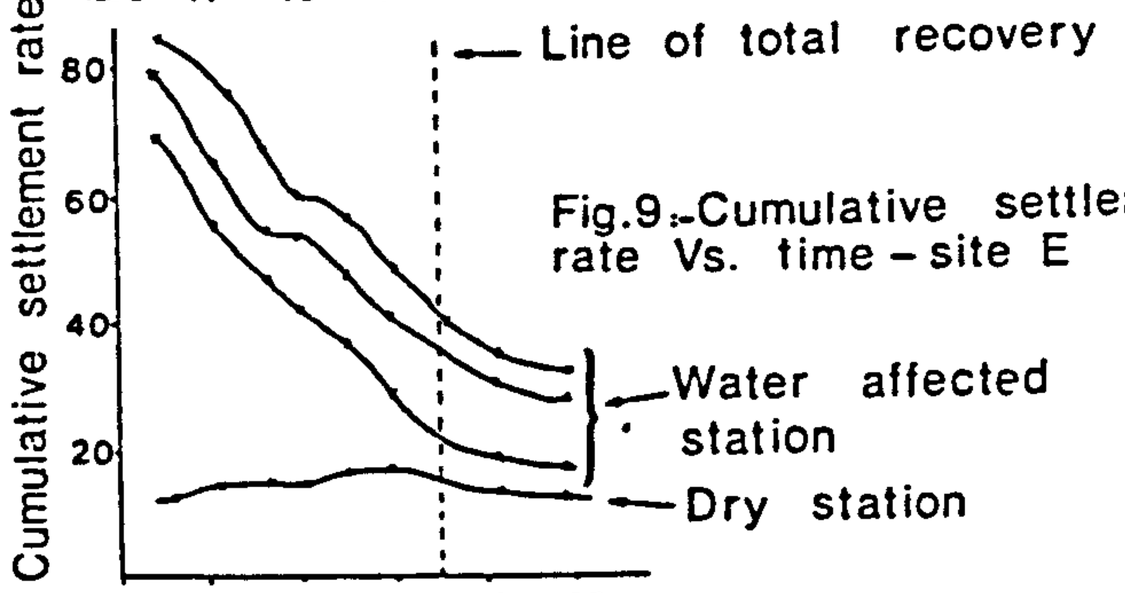

Fig.9 Cumulative settlement rate Vs. time - site E

The results for area A are now presented. Figure 8 details the nature of the groundwater levels within the fill for the period 1975 to 1985, and from 1975 to 1982 the levels can be seen to be recovering, ie full groundwater recovery took seven years to complete. From 1982 onwards the water levels can be seen to fluctuate with a seasonal nature. These levels have been correlated to the positions of the instrumentation for area A, and it has been determined that whilst most of the fill beneath the stations is unsaturated, some stations are lying on fill which has been saturated to 30% of its depth during the recovery period, (c. 30m total depth). The cumulative settlement rates for three of the "water-affected" instruments are presented in figure 9, along with a typical example of an instrument in unsaturated ground along the same traverse.

The three "saturated" stations all show similar trends to each other with an initially high rate of settlement slowly tailing off with time. On the scale of this figure recovery is estimated to have occured, (from the piezometer readings), seven and a half years from the commencement of monitoring, this time being indicated on the figure. Of significant note is the fact that the rates of settlement are reduced to the right of the line. ie post-recovery, indicating that during the recovery phase settlement was being controlled by groundwater and following recovery self weight of the fill became the predominant factor.

The figure also shows a typical trend for a "dry" station, which shows none of the characteristics of those subjected to a water recovery phase. Of note for this particular instrument is that a constant rate of surface settlement has been achieved for a period of 11 years thus indicating the lengthy timespans over which even unsaturated fills may undergo consolidation.

Research into Groundwater Pollution.

The Department has commenced research into the groundwater pollution aspects of groundwater recovery within opencast mine fills. The aims of this work are as follows:

* To estimate the potential pollution loads which may be expected to be generated from an Backfilled Site.
* To observe polluting effect on the local groundwater geochemistry.
* To evaluate methods of combatting a pollution problem.

Laboratory testing includes:

* Chemical analysis of backfill samples to determine acid/alkali forming materials.
* Simulated weathering tests - bacterial leaching.

The research is attempting to correlate:

* Local groundwater geochemistry.
* Levels of water within a fill.
* Material facets of a fill.
* Ages of fill.

This research is still in the initial testing stages and no firm conclusions can yet be drawn, and therefore the results will be reported in due course.

Conclusions.

The following conclusions have been drawn to date from this research project.

* Backfill consolidation can be greatly increased in magnitude and rate by the presence of a recovering water table.

* In the absence of water within a fill, movements may be negligible.

* Following the completion of the sequence of groundwater recovery settlement rates can be significantly reduced.

* Settlement rates of dry backfill materials have been recorded to stay constant for periods of 11 years.

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PAPER C5 SITE INVESTIGATION

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Site investigation in relation to opencast coal mining in Great Britain (Chapter 3).

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NATIONAL COAL BOARD

OPENCAST EXECUTIVE

SITE INVESTIGATIONS

IN RELATION TO OPENCAST COAL MINING IN GREAT BRITAIN

By D. B. HUGHES

SITE INVESTIGATIONS IN RELATION TO OPENCAST MINING IN GREAT BRITAIN

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SITE INVESTIGATIONS IN RELATION TO OPENCAST MINING IN GREAT BRITAIN

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NATIONAL COAL BOARD OPENCAST EXECUTIVE

SITE INVESTIGATIONS

IN RELATION TO OPENCAST COAL MINING IN GREAT BRITAIN

(Notes prepared for the Experienced Engineers course to be held at Graham House, July 1983)

1. INTRODUCTION

1.1 Objects of a Site Investigation

Site investigations are made to determine the ground conditions which affect the design, cost and performance of proposed engineering projects. The main objects are:-

(i) verification, amplification or investigation of the geology of the site, i.e. sequence of strata, both soil and rock, their form, variation and continuity.

(ii) determination of soil and rock properties.

(iii) to assess the usefulness of soil and rock as a material for engineering construction purposes.

(iv) determination of ground water conditions.

This information should make it possible to:-

(v) assess whether the proposed site is capable of withstanding the proposed engineering structure.

(vi) prepare an adequate and economical design.

(vii) foresee and provide against difficulties that may arise during and after construction due to ground conditions.

1.2 The Philosophy of Site Investigations

If you do not know what you should be looking for in a site investigation you are not likely to find much of value. What you should be looking for should be suggested by the natural environment and the nature of the constructional problem to be solved. Thus a detailed programme of investigation cannot be decided on one day and strictly adhered to ..... the number and location of boreholes and trial pits, the number and nature of the tests to be made whether in-situ or in the laboratory should be decided as the work proceeds.
(Glossop-8th Rankine Lecture)

1.3 Engineering Works which usually require a Site Investigation

Some type of site investigation is required where any of the following works or structures are planned:-

(i) TEMPORARY SPOIL MOUNDS AND EXCAVATED SLOPES
- founded on, or dug through, superficial materials.

(ii) EMBANKMENTS AND CUTTINGS
- for new rail sidings, or access roads etc.

(iii) RECLAMATION OF DISUSED SPOIL HEAPS

(iv) FOUNDATIONS
- for loading bunkers, crushing plant, bridges, retaining walls, hoppers and silos, and other large buildings.

(v) WATERCOURSE, AND PIPELINE DIVERSIONS
- both in open-cut (trench) or heading(tunnel).

(vi) CARRIAGEWAYS
- for new access roads, public highway diversions, improvements to existing roads.

The amount and type of site investigation work to be carried out for any particular project is determined by the nature and distribution of the superficial deposits, the nature and depth of bedrock, the existence of old underground workings (if any), the area and depth of ground to be excavated during the works and the area and magnitude of any foundation loadings.

1.4 British Standard Codes of Practice

The following Codes of Practice have been published by the British Standards Institution and relate to site investigation practice.

(i) B.S.5930: CODE OF PRACTICE FOR SITE INVESTIGATIONS

This is a manual of current practice for the British site investigation industry. It deals with ground investigation techniques, sampling, in-situ testing, descriptions of soils and rocks, reporting of results etc.

(ii) B.S.1377: METHODS OF TEST FOR SOIL FOR CIVIL ENGINEERING PURPOSES

All the basic soil mechanics laboratory tests are described in this standard. However, there are more complex tests which are frequently carried out, but which are not covered by B.S.I. publications.

(iii) B.S.6031: EARTHWORKS

This is a manual of current practice for earthworks, including assessment of slope stability.

(iv) C.P.2004: FOUNDATIONS

This is a manual of current practice for foundation design and construction.

1.5 Voluntary Code of Practice for Spoil Mounds

A Code of Practice "The Siting and Construction of Temporary Spoil Mounds at Opencast Coal Sites" has been drawn up by a joint working party nominated by the Federation of Civil Engineering Contractors and the N.C.B. Opencast Executive, and in consultation with H.M. Inspectorate of Mines and Quarries (Health and Safety Executive). The Code is in two parts, Part 1, to be completed by the Opencast Executive's'competent person', and Part 2, to be carried out by a 'competent person' appointed by the Owner (Operating Contractor). The two parts together form Addendum Q of the Contract General Specification.

A summary of the requirements of the Code is as follows:-

PART 1. The O/E competent person has to carry out a site investigation, and from the results produce a site plan and cross-sections of the proposed spoil storage areas. In addition he is required to give an account of the geotechnical nature of the superficial deposits including soil mechanics laboratory test results and strength parameters.

PART 2. The Owner's competent person designs the mounds using the information supplied in Part 1 and applying spoil mechanics design principles. He also designs any necessary drainage works; and specifies the periods between inspections if required to be more frequent than suggested in the Code.

The N.C.B. O/E and the F.C.E.C. have "volunteered" the use of this Code of Practice to the H. & S.E.. Amended "stability" clauses and an Addendum Q have been introduced into the O.E. General Specification. The Code of Practice is now included in all new Contracts for the working of opencast coal.

1.6 Superficial Deposits

Most opencast coal mining regions in Britain have some degree of superficial soil cover over the Coal Measures bedrock, but this varies considerably between the regions. For example, in North East Region about 25% of all excavated materials on opencast sites are classified as superficials and mainly comprise glacial tills (boulder clay, laminated clay, sand and gravel etc.) also some residual soils, alluvial deposits, peat and made ground (colliery spoil and opencast backfill etc.). In Central East Region there is only a very occasional cover of glacial material and the proportion of superficials is much less; but there are still some areas of residual soils, alluvium and made ground.

2. STAGES IN A SITE INVESTIGATION

The carrying out of a site investigation for a particular project may be considered to consist of a number of stages.

2.1 Preliminary Stage

A thorough study of all available geological and geotechnical records (maps, previous investigations done in the vicinity, air-photography etc.), plus a careful walk-over-survey of the site, is always carried out before commencing any borehole or trial pitting operations. This is done to facilitate the planning and optimisation of the main fieldwork investigations.

At this preliminary stage it is also necessary to investigate the land access situation (ownerships, crops, access tracks and gateways, surface conditions etc.) and to obtain location details of all statutory undertakers equipment (underground and overhead services etc.). These factors have a very considerable bearing on when and where site investigation fieldwork may be carried out.

On large projects or sites where very difficult ground conditions are anticipated, it may be desirable to sink a few boreholes as a preliminary to the main fieldwork operations. This should permit a preliminary assessment of the ground conditions and foundation problems, and so lead to a more successful and economical main investigation.

2.2 Main Site Investigation

At this stage the main fieldwork operations (boreholes, trial pits, sampling, in-situ testing), laboratory testing, and application of the results are carried out. These operations are described in detail in Sections 3, 4 and 5 of these notes.

2.3 Foundation Investigation

During excavating for, and construction of engineering foundations, it is usual to compare the ground conditions revealed against the conditions predicted by the main site investigation. If the actual conditions are different, it may lead to a change or modification in the design of the foundations.

In 1948 engineers of the U.S. Bureau of Reclamation referred to the "Design-as-you-go" principle. More recently it has been said that if an engineer is not diligent in continuing to investigate the foundations after they have been opened up and if he is reluctant to review his designs in the light of information obtained after the award of a contract or to admit the need for changes in design, such an attitude can be very dangerous.

(Binnie et al, 1967, Mangla, Proc. I.C.E. Nov. 1967)

In some cases plate loading tests or pile loading tests are carried out to check that the designs are adequate.

2.4 After Construction Investigation

This is mostly carried out with important or very heavy structures, or structures or works in a sensitive situation, and involves instrumenting or monitoring the site or structure. For example, from the results of the main site investigation the total and differential settlements of the foundations of a structure will have been predicted. By carrying out a simple levelling check over a period of time after completion, it will be shown whether the actual settlements are within the permitted tolerances.

3. GROUND INVESTIGATION TECHNIQUES

3.1 General

For coal prospecting purposes the N.C.B. Opencast Executive Development Department employs air-flush rotary openhole drilling, plus some core sampling of the coal seams and of the overall bedrock strata succession. However, in the superficial deposits, this method of penetrating the ground does not yield satisfactory samples for soil mechanics testing purposes or permit detailed engineering descriptions of the soils. Therefore alternative methods of forming exploratory holes are employed for site investigations in superficial materials, e.g. cable percussion boring and trial pits.

Rotary openhole drilling is however very useful for proving overall thicknesses of superficial deposits say over the whole of an opencast site area, or as a preliminary method for assessing alternative sites for new structures. Also in the foundation engineering situation, rotary drilling may be used to penetrate bedrock to prove the presence of underground workings, and to prove the adequacy of the rock as a founding stratum (say for end-bearing piles etc.)

Cable percussion boring is however the most important and common technique used to investigate our superficial deposits. Therefore in the following notes the section on cable percussion boring is the most detailed and extensive.

3.2 Cable Percussion Boring (modern name)
(Shell and Auger Boring - old fashioned name)

3.2.1 BORING

Engineering soils (clays, silts, sands, gravels, made ground) are usually penetrated using the cable percussion boring rig. This method employs the use of drop tools attached to a wire rope or cable, the two basic tools being the 'claycutter' and the 'shell' These are open-ended steel tubes of varying lengths and diameters with specially designed separate cutting shoes. (The rig, plus boring and sampling tools, are illustrated in Figures 1 and 2).

The winch capacities of these rigs vary between 1 and 2 tonnes. The maximum depth of penetration attainable depends upon the nature of the ground being investigated, and to some degree on the skill and experience of the operators. Using several reducing sizes of casing boreholes to 60m. depth are claimed to be possible. However, where glacial deposits are the material being penetrated 30 to 40m. depth seems to be the practical limit.

Each rig is normally operated by a two-man crew, but where very deep holes are being bored necessitating large diameter casing and tools, the additional weight often requires the employment of a third man.

In cohesive (clay) soils the borehole is advanced by the percussive action of the claycutter. The tool falls by gravity to impact on the soil and is raised up again by the winchrope after each impact. In this way, a plug of clay forms in the base of the claycutter. Once this plug is of sufficient size, the spoil is emptied from the boring tool by pushing out with a bar or rod. Another tool often used in clay strata is the cross-blade cutter, the method of operation being similar to that for the conventional claycutter.

For boring in stiff clays and highly weathered rocks the weight of the tool can be increased by the addition of 'sinker bars'. The use of excessively heavy tools in soft clays should be avoided as this is likely to cause disturbance below the base of the borehole.

To penetrate weak rocks, boulders and large cobbles, heavy chisels are used, which can weigh up to 250 kg. Progress is slow as the material has to be removed with a shell after it has been broken up by the chisel.

When boring in soft clays the borehole is unlikely to remain open without the support of steel lining tubes or casing. Although the hole may not collapse completely, the tendency is for the sides to squeeze inwards and jam the boring tools. The usual practice in such clays is for the operator to bore ahead of the steel casing for a distance of 1.5m. (the standard length of a casing section) before adding a new section of casing and surging it down. The reason for surging the casing is to keep it 'free' in the borehole so that it can be easily extracted on completion. When the operator can no longer advance the casing by surging he will reduce to a smaller diameter with the advantage of having a length of the smaller size casing free-standing inside the larger hole. This reduces the frictional resistance of the casing and the boring progress generally increases. The decision to reduce to a smaller size is not taken lightly. The operator has to decide whether the reduction in boring time justifies the delays incurred in running-in the extra set of casing and extracting it on completion. If the hole is near its final depth he may resort to 'driving' the casing with the claycutter and 'jacking-out' the casing on completion.

In stiff clays the borehole can very often be advanced without lining tubes, and within the time required to make the boring the borehole often remains dry. A short length of casing is used at the top of the boring to keep the hole stable and prevent collapse. Where clays occur below granular deposits the casing (used as a support in the granular soils) is driven a short distance in the clay to create a seal and the shell is used to remove any water which might enter the borehole. In very stiff clays a little water is often added to assist boring progress. This must be done with caution to avoid possible changes in the properties of the soil to be sampled.

In granular soils (silts, sands and gravels) the shell (also called the bailer) is used to advance the borehole. This differs from the conventional claycutter in that a clack valve is fitted just above the cutting shoe. This valve ensures that the soil which is forced into the shell by the percussive boring action is retained until the tool is lifted out of the borehole and is emptied to one side.

Sands and gravels nearly always require the use of lining tubes to support the sides of the borehole. The casing must be advanced with the borehole or the continual collapse of the sides of the borehole below casing will prevent further progress and result in a cavity being formed (a condition known as 'overshelling'). Cases have been known where overshelling has caused a depression to occur at ground level. The piston action of the shell causes a loosening of the material at the base of the borehole and the casing is driven into this disturbed zone. Because of the mode of operation of the shell the borehole must be full of water for the shell to operate efficiently. Since most granular strata in Britain are water bearing all that is required is a supply of water to keep the natural water level in the borehole 'topped up'. The flow of water must always be from the borehole into the surrounding strata. If this condition is reversed by allowing water to flow into the borehole, 'piping' will probably occur, and will invalidate the results of standard penetration tests (see later). 'Piping' is the term used to describe the condition where material is carried up the borehole when the hydrostatic head in the borehole is less than the hydrostatic head in the surrounding ground. Provided the head of water in the borehole is greater than in the surrounding ground, piping can usually be prevented. To overcome slight artesian heads the lining tubes must be extended above ground level and kept filled with water. Larger artesian heads would require the rig to be raised up on a platform, but this is very rarely necessary.

The importance of good groundwater recording cannot be overstated, and the following rules are regarded as a minimum:-

(i) Record depth of borehole and casing for all water entries, including rate of entry.

(ii) Record depth where water entry is sealed by casing.

(iii) Record standing water levels, plus borehole and casing depths, at start and finish of each working shift.

(iv) Adding of water to assist boring should be kept to an absolute minimum, and if used should be recorded with an explanation as to why it was used.

For longer term observation of groundwater levels, standpipes and piezometers are installed into boreholes and these are described later.

3.2.2 SAMPLING

(i) Disturbed Samples

Small representative disturbed (jar) samples of about 0.5 to 0.7 kg. from the clay-cutter or shell and sealed in glass or plastic jars. These are chiefly used as recognition samples, but may be used for moisture content and index testing purposes.

Large or 'bulk' samples (usually about 30 kg.) of coarse granular materials are also taken from the boring tools and placed into strong polythene sacks. Where such samples are intended for particle size analysis (gradings) the following procedure is adopted to prevent loss of 'fines'. The whole contents of the 'shell' (repeated 2 or 3 times) is emptied into a tank, then after allowing to settle the water is poured off and the material mixed by hand before taking a representative sample.

(ii) Undisturbed Samples

The normal undisturbed sample used in stiff cohesive soils is the U100 tube sample which is nominally 450 mm. long x 100 mm. dia., is fitted with a cutting shoe and is driven into the soil below the casing using a jarring-link (sliding hammer arrangement). On removal from the borehole the ends of the U100 tube are sealed with molten paraffin wax, and then capped with steel or plastic end caps to retain the moisture in the sample. In alluvial clays and other softer or more sensitive clays, thin-walled piston samples are preferable. However, these would be too easily damaged if used in most glacial materials.

(iii) Groundwater Samples

These are usually taken from the 'shell', and retained in a jar of about 1 litre capacity

3.2.3 IN-SITU TESTING

(i) Standard Penetration Test (S.P.T.)

In this test a standard split spoon sampler 50mm. dia. is driven through the soil at the bottom of borehole for a distance of 450mm. and the number of blows of a 64 kg. weight falling through 760mm. needed to drive the spoon sampler through the last 300 mm. of soil is recorded as the S.P.T. 'N' value.

The spoon sampler usually retains a small sample of the soil penetrated, and this can be placed in a jar as a 'representative disturbed sample'. In coarse gravels or hard bedrock the spoon sampler is usually fitted with a solid cone (C.P.T.) to prevent damage, but no sample is then recovered.

Empirical relationships between N, relative density, angle of shearing resistance, allowable bearing capacity and settlement have been obtained - see textbooks on Foundation Engineering.

(ii) Permeability Tests.

The determination of in-situ permeability by tests in boreholes involves the application of a hydraulic pressure in the borehole different from that in the ground, and the measurement of the flow due to this pressure difference. There are several methods of test including rising head, falling head, constant head and variable head, and the methods for each are fully described in BS.5930. These tests are not carried out in the majority of site investigations, and their main application is in investigating excavation dewatering problems

3.2.4 TYPICAL IN-SITU TESTING AND SAMPLING ROUTINE

(i) Cohesive Soils

U100 tube sample every 1.0m. for the first 10m. of boring, then every 1.5m. minimum thereafter, and after each change in stratum. A jar sample should be taken between each U100 sample.

(ii) Non-cohesive Soils

S.P.T. (or C.P.T.) every 1.0m. for the first 10m. of boring, then every 1.5m. minimum thereafter, and after each change in stratum. The S.P.T. sample should be retained in a jar, or for C.P.T.'s a separate jar sample should be taken. Also bulk samples suitable for grading tests should be taken between each S.P.T. or C.P.T.

(iii) Bedrock

S.P.T.'s or C.P.T.'s every 1.0m., including a test on completion of the borehole. Bedrock should be proved by chiselling for 1 metre or 2 hours, whichever is achieved first.

(vi) Groundwater.

1 litre samples from all significant water strikes.

(v) General

All S.P.T.'s or C.P.T.'s and U100 samples must be taken in the soil below the level of the casing, otherwise the test or sample will be in disturbed ground and therefore not valid.

Where an existing slope failure in clay strata is being investigated, continuous U100 tube sampling may be adopted. The samples can be extruded in the laboratory and examined to find the actual failure plane.

3.2.5 STANDPIPES AND PIEZOMETERS

In order to observe the variation of water level in the ground over a period of time standpipes or piezometers are installed in a completed borehole

(i) Standpipes (Figure 3)

Steel or P.V.C. pipes of 12 to 25 mm. dia., fitted with a porous plastic or ceramic filter tip on the lower end, are placed down the cased borehole. As the casing is withdrawn the borehole is backfilled with fine gravel or coarse sand to within 1 metre of the surface, and thereafter with impervious material to prevent the ingress of surface water. A vented protective cover pipe is then provided at the surface. An electronic dipmeter is used to measure the water levels.

(ii) Piezometers (Figure 4)

When measurements of head of water are required in a particular stratum piezometers may be installed. Piezometers differ from standpipes only in that their active tips are sealed within the particular stratum. Permeability tests can be carried out in piezometers, and the procedures are described in BS.5930.

3.2.6 BOREHOLE REPORTS

These are often referred to as borehole logs, and are usually plotted to a vertical scale. A typical example for a complete cable percussion borehole with piezometer installed is given in Figures 5, 6 and 7

3.3 Trial pits

Trial pits are a cheaper method than cable percussion boring for investigating to shallow depths. Excavation can be by hand in areas where access is very difficult, otherwise it is preferable to use a small mobile excavator (backhoe). The general depth limit is 5m. in clays, but in water-bearings soils (sand, peat, etc.) the sides of the pit may slump down and inspection and sampling becomes difficult or impossible.

If it is necessary for men to enter the pits, say for logging or sampling purposes, then support of the sides of any pits deeper than 1.2m. will be necessary if there is a risk of collapse.

In the right ground conditions, trial pits enable a detailed picture to be obtained of the stratification and fabric of the soils. They also enable hand-cut (Block) samples to be taken, as well as undisturbed tube (U100) and large disturbed samples.

The best application of trial pits is probably in highway investigations (for carriageways and shallow cut or fill) as they permit the obtaining of large samples required for compaction and C.B.R. testing.

3.4 Hand augering

The hand auger boring method uses light hand-operated equipment. The auger and drill rods are usually hand rotated into the ground and lifted out of the borehole without the aid of a tripod, and no casing is used. Augers are usually 150mm. or 200mm. diam. and holes can be sunk to as deep as 5m. in soft to firm clays. Water-bearing soils (sands, peat) will not stand open, and soils containing particles larger than coarse gravel size cannot be penetrated. Small disturbed samples and small open-tube (37mm. dia.) samples can be obtained.

3.5 Rotary Drilling

Rotary open hole drilling and rotary core drilling are the normal methods of drilling through rock in which the drill bit is rotated on the bottom of the borehole. The drilling fluid, which is pumped down to the bit through hollow drill rods, lubricates the bit, and flushes the drill debris up the borehole. The drilling fluid is most commonly water, but air or drilling mud may be used.

There are two basic types of rotary drilling; open hole drilling, in which the drill bit cuts all the material within the diameter of the borehole; and core drilling, in which an annular bit fixed to the bottom of the outer rotating tube of a rotary core barrel cuts a core, which is retained within the inner stationary tube of the core barrel and brought to the surface for examination and testing. Drill casing is normally used to support unstable ground or to seal off open fissures which cause a loss of drilling fluid. Alternatively, drilling mud or cement grouting can be used. Rotary drilling for ground investigation is usually core drilling.

Core barrels are available in various types and sizes, usually between 17mm and 165mm. in diameter and between 1.5m. and 3.0m. in length. Coring bits are usually hardened steel impregnated with industrial diamonds, or sometimes for use in softer rocks they are inset with tungsten carbide teeth.

Machines for rotary drilling are either lorry mounted, trailer mounted or skid-mounted. Also, for extending a cable percussion bored hole for a few metres into bedrock, pendant rotary attachments are available for coring up to 54 mm. dia.

3.6 Indirect Methods (Geophysics)

Indirect methods of exploration by geophysical methods are sometimes of use on large construction projects because of their relative speed. These methods can be used for determining the boundaries between strata of different composition and also the position of the water table. Some knowledge of the composition of the strata may be inferred but generally geophysical techniques must be supplemented with boreholes.

The main methods of investigation are

(i) seismic method - using the elasticity and density of the ground rock
(ii) electrical resistivity - using the conductivity of the ground
(iii) proton magnetometer - using magnetism
(iv) gravimeter - using density

The use of geophysical methods is fairly rare in connection with site investigations for works connected with opencast mining. However geophysical logging of coal prospecting boreholes is carried out extensively.

More detailed descriptions of the various geophysical methods can be found in BS 5930.

CABLE PERCUSSION RIG
Pilcon Wayfarer 1500

FIGURE 1

GENERAL
Percussive drilling rig mounted on two wheels for towing. The shear legs form the tow bar and the rig is self erecting using its own winch.

SPECIFICATION

CHASSIS & LEGS
Electrically welded from steel tube and sections.
Designed load of legs: 7000kg.

WHEELS & SUSPENSION
550 x 16 Land-Rover wheels with 650 x 16 8-ply tyres carried on 'Avonride' rubber suspension units.

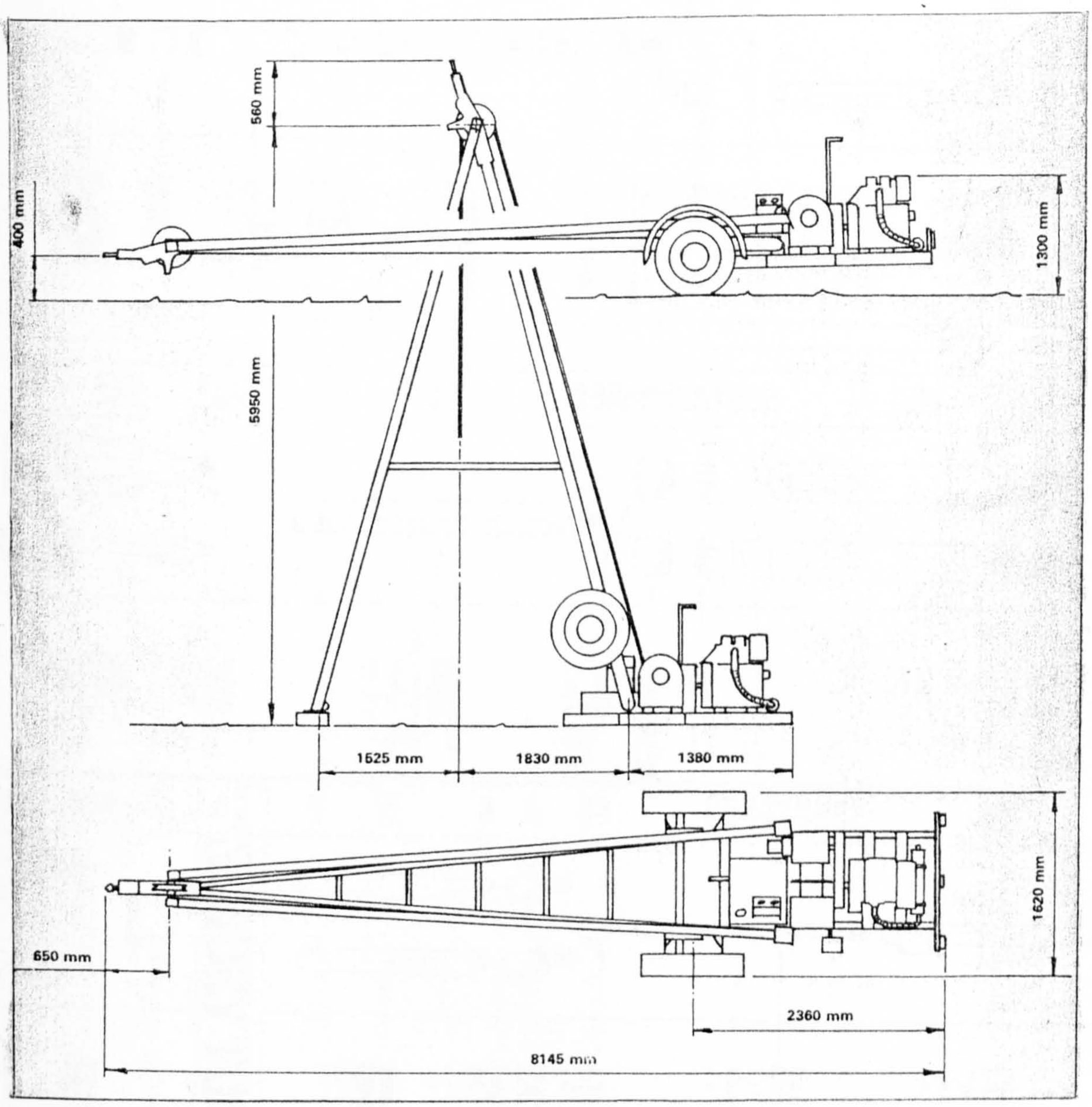
ENGINE
Petter BA2 lightweight twin cylinder diesel, developing 15kW (20 hp) at 3000 rpm.
(Recommended maximum speed 2500 rpm, giving 18 hp continuous output.)
Vee belt drive to winch.

WINCH
Free-fall type, belt driven from engine. Worm reduction gearbox, driving a Meehanite rope drum with integral brake and clutch surfaces. Band brake controlled by foot lever with a raised fine adjustment locking lever.

CLUTCH
Either Pilcon hydraulic actuated clutch (Model 36.02). or twin sintered disc mechanically operated clutch (Model 36.02A).

OPTIONS
Overriding brakes, lights, mudguards.
Automatic spring starter.
Electric starter.
Lockable steel cover.
Hydraulic power take-off.
Output flow: 35 litre (7·75 gpm).
Pressure: 17MN/m² (2500 psi).

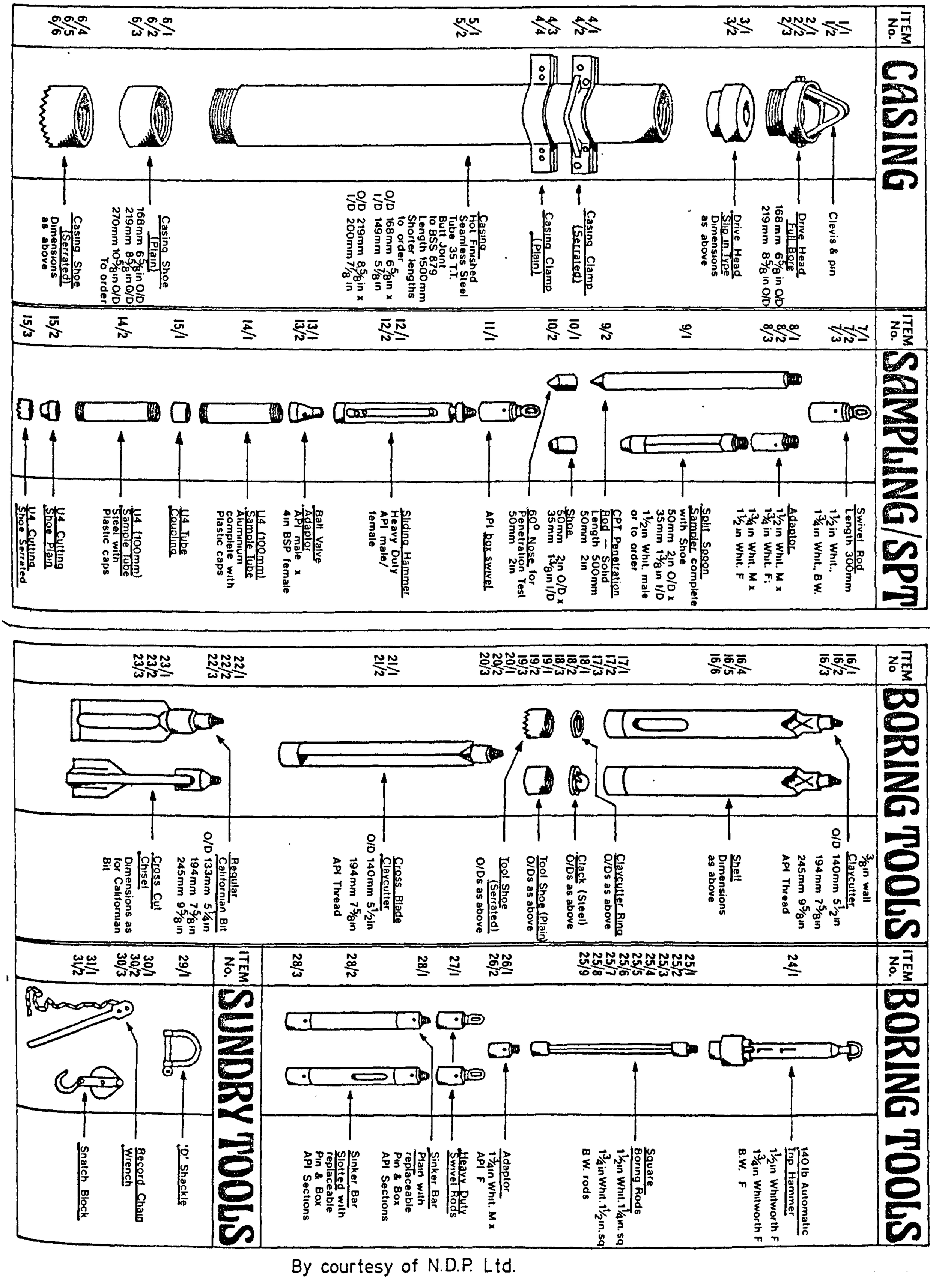
SHIPPING DIMENSIONS
(Legs detached from chassis.)
WEIGHT – 1120kg (2460lb) Approx.
LENGTH – 6600mm
WIDTH – 1620mm
HEIGHT – 1100mm



By courtesy of Messrs. Pilcon

CABLE PERCUSSION BORING TOOLS
by Northumbrian Drilling Products Ltd.

FIGURE 2



By courtesy of N.D.P. Ltd.

FIGURE 3

STANDPIPE INSTALLATION DETAILS

NOT TO SCALE

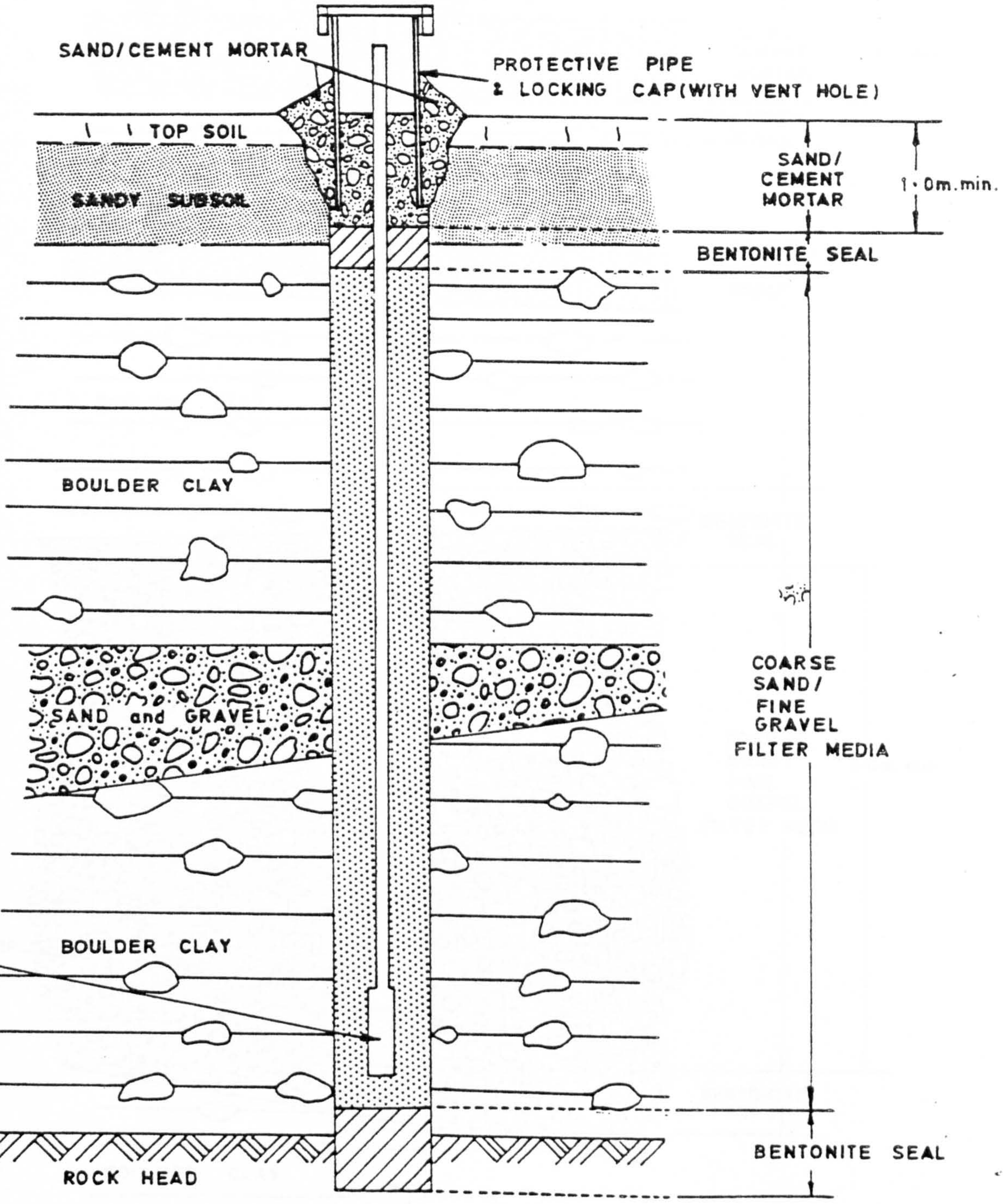
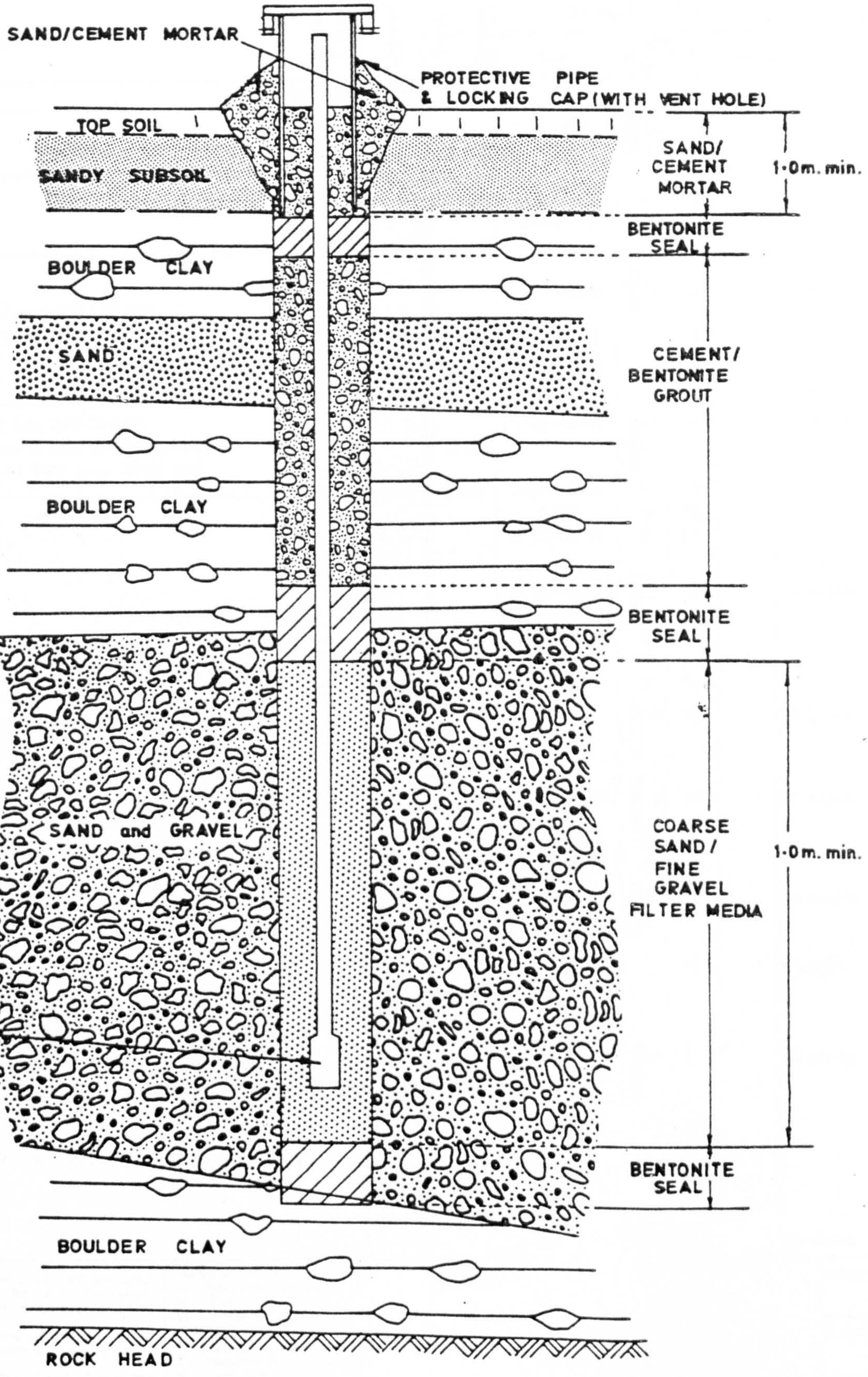


FIGURE 4

PIEZOMETER INSTALLATION DETAILS

NOT TO SCALE



Soil Mechanics

Borehole Log Sheet
FIGURE 5

Borehole No. 6
Sheet 1 of 3

Equipment & Methods
Dando 150 cable tool boring 250mm dia.
GL to 23.00m

Location No. 7693/2
BLACKBOY, CO. DURHAM

Carried out for
National Coal Board

Ground Level
104.48m OD

Coordinates
423542E; 528046N

Date
1–8.5.80

Description	Reduced Level	Legend	Depth & Thickness	Samples/Tests: Depth	Sample: Type	Sample: No.	Test	Field Records
Topsoil	104.38		0.10					
Soft brown and orange mottled very sandy silty CLAY			(1.35)	0.30–0.65	U	1		10 blows
				0.65–0.75	D	2		
				1.00–1.30	B	3		
				1.45–1.55	D	4		
	103.03		1.45					
Soft to firm thinly laminated grey silty CLAY with occasional thin sandy laminations and trace of fibrous peat			(1.25)	1.60–2.05	U	5		15 blows
				2.05–2.15	D	6		
				2.30–2.60	B	7		
	101.78		2.70					
Grey brown silty coarse to fine SAND with trace of gravel sized fragments of coal and occasional thin laminations and beds of soft grey silty clay			(0.80)	2.90–3.35	U	8		15 blows
	100.98		3.50	3.35–3.45	D	9		
				3.50	W	10		
				3.70–4.00	B	11		
				4.20–4.65	U	12		12 blows
				4.65–4.75	D	13		
				5.00–5.30	B	14		
Soft to firm grey brown slightly sandy silty CLAY interbedded with grey and brown SAND with trace of gravel			(5.10)		B	15		25 blows
				5.90–6.35	U	16		31 blows
				6.35–6.45	D	17		
				6.45–6.80	B	18		
				5.80–6.25	U	19		24 blows
				7.60–7.90	B	20		Backfill
Trace of sand sized fragments of coal	95.88		8.60	8.20–8.65	U	21		32 blows
Grey brown slightly clayey silty medium to fine SAND			(1.65)	9.00–9.30	B	22		
				9.70–10.15	U	23		25 blows

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N–value).

Depths: All depths and reduced levels in metres. Thicknesses given in brackets in depth column.

Water: Water level observations during boring are given on last sheet of log.

Sample/Test Key.
- D Disturbed Sample
- B Bulk Sample
- W Water Sample
- ▮ Piston (P) Tube (U) or core sample : Length to scale
- S Standard Penetration Test
- V Vane Test
- C Core recovery (%)
- r Rock Quality Designation (RQD–%)

Remarks

Logged by LT

Scale 1:50

Fig. 5

ϕ Soil Mechanics

Borehole Log Sheet
FIGURE 6

Borehole No. 6
Sheet 2 of 3

Equipment & Methods	Location No. 7693/2
As sheet 1	BLACKBOY, CO. DURHAM
Carried out for	**Ground Level** **Coordinates** **Date**
National Coal Board	As sheet 1

Description	Reduced Level	Legend	Depth & Thickness	Samples/Tests: Depth	Sample: Type	Sample: No.	Test	Field Records
Medium to fine SAND (As sheet 1)	94.23		10.25	10.15–10.25	D	24		
				10.35	W	25		
				10.65–10.95	D	26	S N=7	Bentonite seal
				11.00–11.35	B	27		
				11.65–11.95	D	28	S N=8	
				12.00–12.35	B	29		Tip at 12.50m ▲
				13.15–13.45	D	30	S N=9	
Loose grey brown coarse to fine SAND with occasional silty layers and thin lenses of sand and fine gravel sized fragments of coal				13.90–14.30	B	31		
			(10.35)					
				14.75–15.05	D	32	S N=10	
				15.30–15.70	B	33		
				16.15–16.45	D	34	S N=3	
				16.90–17.20	B	35		
								Backfill
				17.65–17.95	D	36	S N=9	
				18.20–18.60	B	37		
Trace of gravel				19.35–19.75	D	38	S N=31	
				19.80–20.00	B	39		

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N – value).

Depths: All depths and reduced leveis in metres. Thicknesses given in brackets in depth column.

Water: Water level observations during boring are given on last sheet of log.

Sample/Test Key.
- D Disturbed Sample
- B Bulk Sample
- W Water Sample
- ┃ Piston (P) Tube (U) or core sample : Length to scale
- S Standard Penetration Test
- V Vane Test
- C Core recovery (%)
- r Rock Quality Designation (RQD %)

Remarks 1. Insitu permeability test in borehole. ▲

Logged by LT

Scale 1:50

Fig. 6

Soil Mechanics

Borehole Log Sheet
FIGURE 7

Borehole No. 6
Sheet 3 of 3

Equipment & Methods
As sheet 1

Location No. 7693/2
BLACKBOY, CO. DURHAM

Carried out for
National Coal Board

Ground Level **Coordinates** **Date**
As sheet 1

Description	Reduced Level	Legend	Depth & Thickness	Samples/Tests: Depth	Sample: Type	Sample: No.	Test	Field Records
Loose grey brown coarse to fine SAND (As sheet 2)	83.88		20.60	20.50–20.95	B	40		60 blows
Grey slightly silty, slightly clayey coarse to fine subangular GRAVEL with COBBLES			(0.60) 21.00	20.65	W	41		
				20.70–21.00	B	42		
Very highly weathered, grey silty MUDSTONE in part reduced to clay			(1.80)	21.45–21.75	D	43	S	
				21.70–22.00	B	44	N=59	
				22.15–27.45	D	45	S	
				27.40–22.75	B	46	N=66	
	81.68		22.80	22.95–23.06	D	47	S (88)	
Highly weathered dark grey very weak carbonaceous silty MUDSTONE	81.42		(0.26 pen.) 23.06					
END OF BOREHOLE AT 23.06m								

Water Level Observations During Boring

Date	Time	Depth of Hole m	Depth of Casing m	Depth to Water m	Remarks
1.5.80		3.00	2.80	2.65	Slight seepage
2.5.80	0800	5.00	3.80	2.65	
		10.60	10.40	5.90	Seepage from 10.00m
6.5.80	1800	20.50	20.40	11.70	
7.5.80	0800	20.50	20.40	11.70	
8.5.80	0800	22.00	21.90	11.90	

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N – value).

Depths: All depths and reduced levels in metres. Thicknesses given in brackets in depth column.

Water: Water level observations during boring are given on last sheet of log.

Sample/Test Key.
- **D** Disturbed Sample
- **B** Bulk Sample
- **W** Water Sample
- **I** Piston (P) Tube (U) or core sample : Length to scale
- **S** Standard Penetration Test
- **V** Vane Test
- **C** Core recovery (%)
- **r** Rock Quality Designation (RQD – %)

Remarks 2. Chiselling 6.5 hours from 20.50–22.80m.

Logged by LT

Scale 1:50

Fig. 7

4. SOIL MECHANICS LABORATORY TESTING

Detailed descriptions of most of the soil mechanics testing procedures relevant to the slope stability, foundation engineering and highway engineering, can be found in the following publications:-

(i) BS 1377 Methods of test for soil for Civil Engineering Purposes.

(ii) Akroyd, T.N.W. Laboratory testing in soil engineering, Soil Mechanics Ltd, 1951.

(iii) Croney, D and Jacobs J.C. The frost susceptibility of soils and rock materials, D. of E., T.R.R.L. Report LR90 (Interim revision SR318), Crowthorne.

(iv) Bishop, A.W. and Henkel D.J. The measurement of soil properties in the triaxial test, Edward Arnold 1957.

(v) Wilson, N.E. Laboratory vane shear tests and the influence of pore water stresses, A.S.T.M. Special Technical Publication, No. 361, 1963, 377-385.

(vi) Bishop A.W. A large shear box for testing sands and gravels, Proc. 2nd Int. Conf. Soil Mech & Found. Eng. 1, Sub-sect. 11e, Paper 11e, 207-211.

There are a great many other references on soil mechanics testing and a full list is included in BS 5930.

The following table is a summary of the tests most commonly used in site investigations for opencast projects, and is a shortened version of Table 4 from B.S. 5930.

SUMMARY OF SOIL MECHANICS LABORATORY TESTS (after BS:5930)

Category of test	Name of test	Where details can be found	Remarks
Soil Classification Tests	Moisture content	BS 1377	Frequently carried out as a part of other soil tests. Read in conjunction with liquid and plastic limits, gives an indicatio of the shear strength of cohesive soil.
	Liquid and plastic limits (Atterberg limits)	BS 1377	Used to classify cohesive soil and as an aid to classifying the fine fraction of mixed soil.
	Specific gravity	BS 1377	Used in conjunction with other tests, such as sedimentation and consolidation. Values commonly range between 2.55 and 2.75, and a more accurate value is required for air voids determination. Only occasional checks are needed for most British soils for which a value of 2.65 is assumed unless experience of similar soils shows otherwise. However, determination of specific gravity may be necessary where spoil heap material is concerned.
	Particle size distribution: (a) sieving (b) sedimentation	BS 1377	Sieving methods give the grading of soil coarser than silt. The proportion passing the finest sieve represents the combined silt/clay fraction. The relative proportions of silt and clay can only be determined by means of sedimentation tests which should be carried out when there is a real need for this information.
Soil Chemical Tests	Sulphate content of soil and ground water	BS 1377	The tests assess the aggressiveness of soil or ground water to buried concrete. (See remarks on test of pH value).
	pH value	BS 1377	To measure the acidity or alkalinity of the soil or water. It is usually carried out in conjunction with sulphate content tests. This test and the one above should be performed as soon as possible after the samples have been taken.
Soil Compaction Tests	Dry density of soil on site	BS 1377	Measures the mass of solids per unit volume of soil. Most of these tests are used to establish the dry density of soil, either naturally occurring or compacted fill, to which direct access may be obtained. Some, however, can be applied to samples obtained from depth, and these tests are used when the density of the soil is required in conjunction with other tests.
	Dry density/ moisture content	BS 1377	This test indicates the degree of compaction that can be achieved at different moisture contents.
Pavement design tests	California bearing ratio (CBR)	BS 1377	This is an empirical test used in conjunction with the design of flexible pavements. The test can be made either in situ, or in the laboratory.
	TRRL frost heave test	Croney and Jacobs	A laboratory test used to determine the susceptibility to frost heave of a specimen of compacted soil.
Soil Strength Tests	Triaxial compression: (a) undrained (b) undrained with measurement of pore water pressure (c) consolidated undrained (d) consolidated undrained with measurement of pore water pressure (e) consolidated drained (f) multi-stage triaxial test	BS 1377)))))Akroyd))Bishop and Henkel))	By far the most commonly used of these tests is the standard undrained test. There is a large amount of experience in its use and many partly empirical methods are available to utilize the parameters so obtained in the design of foundations and other sub-structures. The remaining tests also have their own uses which will be found fully described in the references quoted. The tests are normally carried out on nominal 100 mm or 40 mm diameter specimens, as appropriate. Several techniques have been used for both drained and undrained tests, details of which will be found in the references. The test is useful where there is a shortage of specimens, and its main use is with 100 mm nominal diameter specimens, only one of which can be prepared from each sampling tube.

Category of test	Name of test	Where details can be found	Remarks
Soil Strength Tests	Unconfined compressive strength	BS 1377	This simple test is a rapid substitute for the undrained triaxial test, although it is suitable only for saturated non-fissured cohesive soil.
	Laboratory vane shear	Wilson	For soft clay, an alternative to undrained triaxial test where the preparation of the specimen sometimes has an adverse effect on the measured strength of the soil.
	Direct shear box: (a) immediate (b) consolidated immediate (c) drained	) Bishop.) Akroyd.)	In the measurement of the shearing resistance of soil, these tests are an alternative to triaxial tests, although the latter have now largely superseded them. One of their main disadvantages is that drainage conditions cannot so easily be controlled. Another is that the shear is pre-determined by the nature of the test. One of their advantages is that specimens of non-cohesive soil can be more readily prepared than in the triaxial test.
	Residual shear strength: (a) multiple reversal shear box (b) triaxial test with pre-formed shear surface (c) shear-box test with pre-formed shear surface (d) ring shear test	) Many references, see list in BS:5930	The residual shear strength of clay soil is increasingly used in slope stability problems. The multiple reversal shear box test is the one which is most commonly used, although the ring shear test would appear to be the more logical. This latter test tends to give lower parameters than the former.
Soil Deformation Tests	Consolidation: (a) one-dimensional consolidation properties (oedometer test) (b) triaxial consolidation	BS 1377 Akroyd. Bishop and Henkel	These tests yield soil parameters from which the amount and time scale of settlements can be calculated. The simple oedometer test is the one in general use and although reasonable assessment of settlement can be made from the results of the test, estimates of the time scale have been found to be extremely inaccurate with certain types of soil. This is particularly true of clay soil containing layers and partings of silt and sand, where the horizontal permeability is much greater than the vertical. An alternative is to obtain values of the coefficent of consolidation, C_v, from in situ permeability tests and combine them with coefficients of volume decrease, M_v, obtained from the simple oedometer test.
Soil Permeability Tests	Constant head permeability test Falling head permeability test	) Akroyd.	The constant head test is suited to soils of permeability roughly within the range 10^{-4} m/s to 10^{-2} m/s. For soils of lower permeability, the falling head test is applicable. For various reasons, laboratory permeability tests often yield results of limited value and in situ tests are generally thought to yield more reliable data.
	Triaxial permeability test	Bishop and Henkel	

A number of laboratory test result sheets have been included with these notes to illustrate the form of the results and the way in which soil mechanics parameters are plotted.

Norwest Holst Soil Engineering Ltd.

Contract No. F4736 Client N.C.B. (Opencast) Northern Region Location ROSE HILLS O.C.C.S.

SAMPLE DETAILS			CLASSIFICATION										CHEMICAL			DENSITY		COMPACTION			C.B.R	STRENGTH			CONSOLIDATION	
No. & Type	Depth m	Description	w %	L.L. %	P.L. %	P.I. %	<425μ %	S.G.	Particle size distribution Clay %	Silt %	Sand %	Grvl. %	SO_3 %	pH	Org %	Bulk γ Mg/m³	Dry γd Mg/m³	Ram kg	Max γd Mg/m³	O.M.C %	Mean %	Type	c kN/m²	Ø degree	$p_c \rightarrow p_o + 100KN/m^2$ m_v m²/MN	c_v m²/yr
B	0.30	GRAVEL COBBLES and BOULDERS in a silty sandy clay matrix.	25	37	21	16	55		8	27	34	31														
D	2.10	Stiff mottled silty sandy CLAY with some gravel.																								
D	2.40	Brown very silty slightly clayey fine SAND.	16								64	1														
B	3.00		21						9	26																
U	5.00	-Ditto - with silty clay bands and rare gravel.	13.	25	14	11	87		5	18	74	3				1.91	1.69					TU	22 (116	19 0)		
B	6.50	Brown very silty slightly clayey fine SAND.	19								60	0														
B	7.50		21						10	30																
U	8.80	Stiff grey-brown silty sandy CLAY with some gravel and occasional cobbles and boulders.	11	25	14	11	89									2.15	1.94					TU	100 (161	11 0)		
U	10.00	- Ditto -	14	22	18	4	77									2.21	1.94					TEU	0	28		
D	10.70		12																							
U	11.70	- Ditto - sample partially disturbed	12	23	14	9	91															TU	18	0		
B	12.10		12																							
D	13.50	Stiff grey silty CLAY with abundant clayey siltstone fragments.	17																							
B	13.70	Very weak to moderately weak grey clayey SILTSTONE.	21																							
B	14.10																									
W		Water											0.8 p.p. 10^5	7.5												

NOTES
U — Undisturbed
B — Bulk
D — Disturbed

N.P. — Non Plastic

T — Triaxial compression U — Undrained
E — Effective stress W — Unconfined
R — Remoulded S — Shear Box

(-) Fo value for Br-B⁰

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Summary of Laboratory Test Results

Figure 8

Borehole No. 17

PLASTICITY CHART (after TRRL Report No. 1030) FIGURE 9

Used to classify cohesive soils from Liquid and Plastic Limit Test results

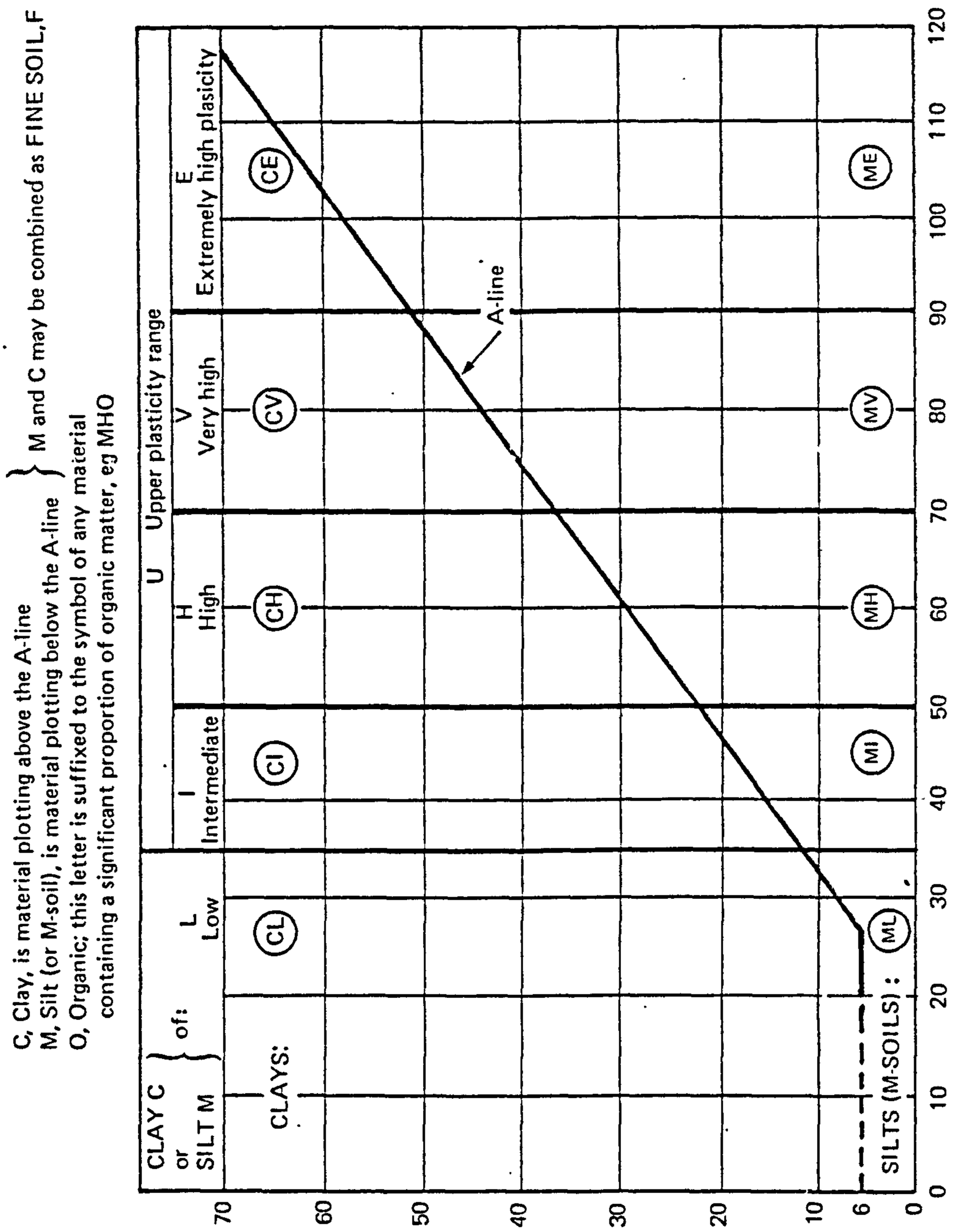
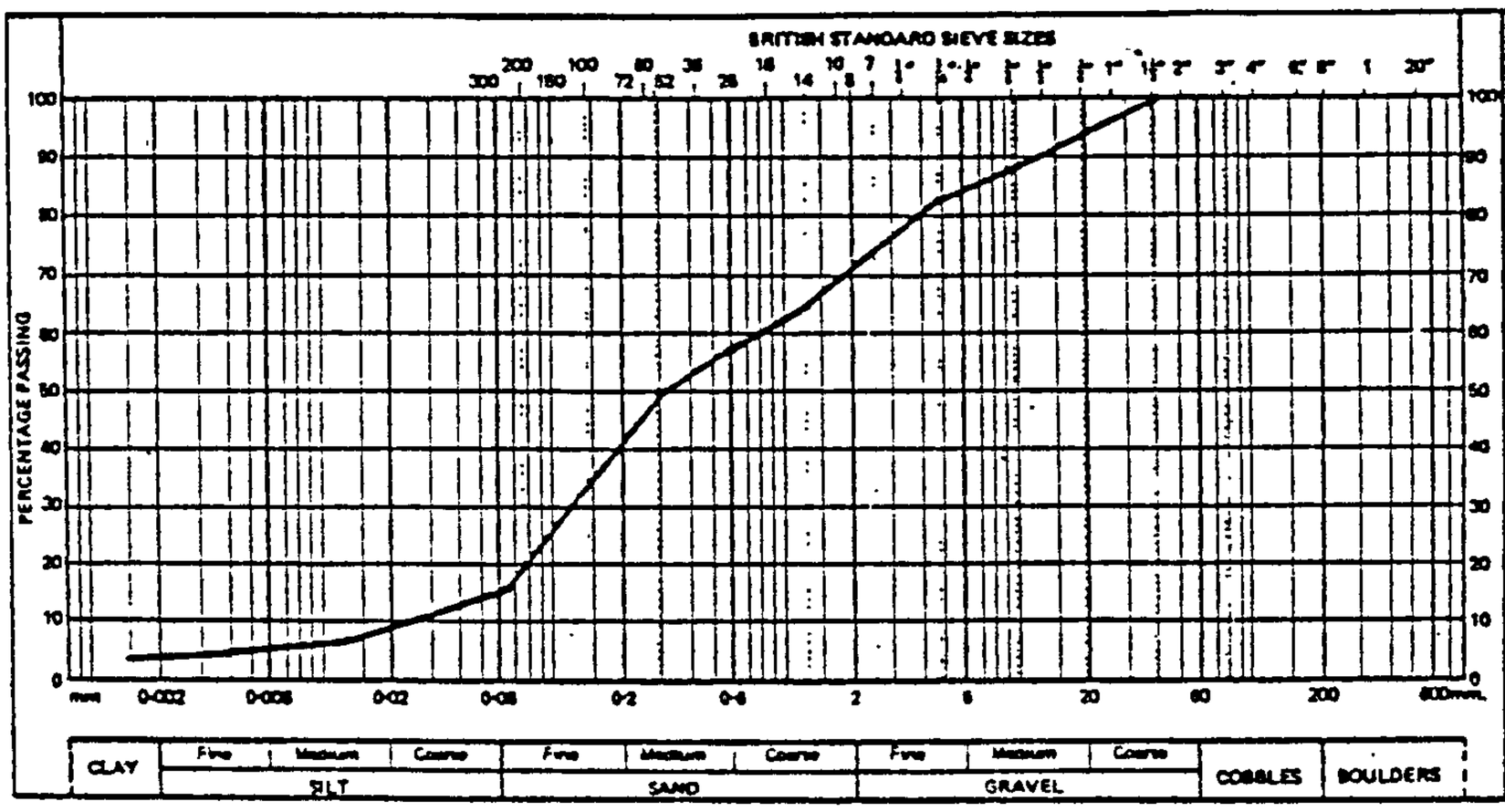


Fig. 1 The plasticity chart—for the classification of fine soils and the finer part of coarse soils (measurements made on materials finer than 425 μm)

PARTICLE SIZE DISTRIBUTION CURVES

FIGURE 10



(a)

Borehole No. 8

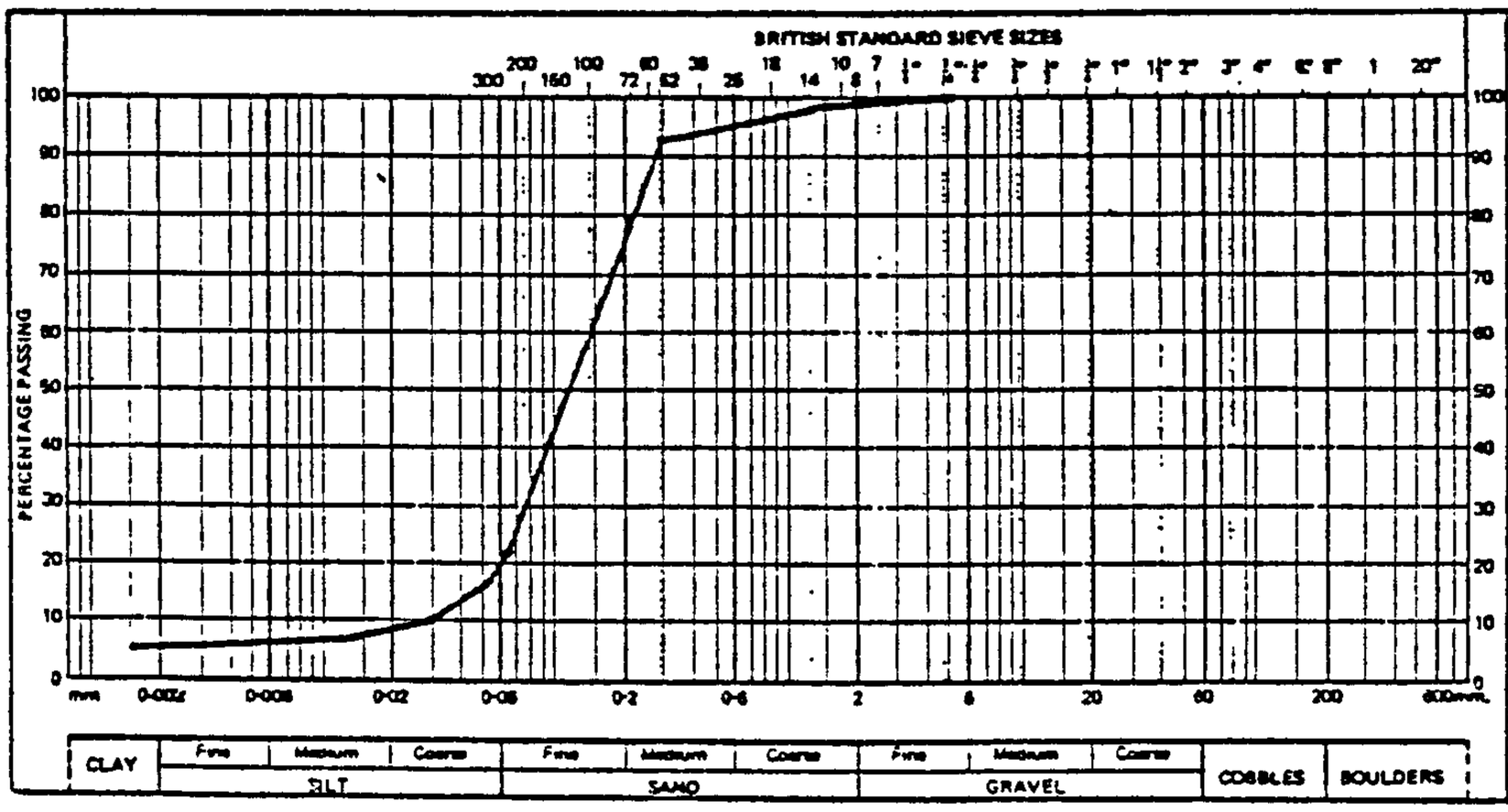
Sample No. 11

Depth: 5.3 m

Remarks

bulk sample

gravelly SAND



(b)

Borehole No. 8

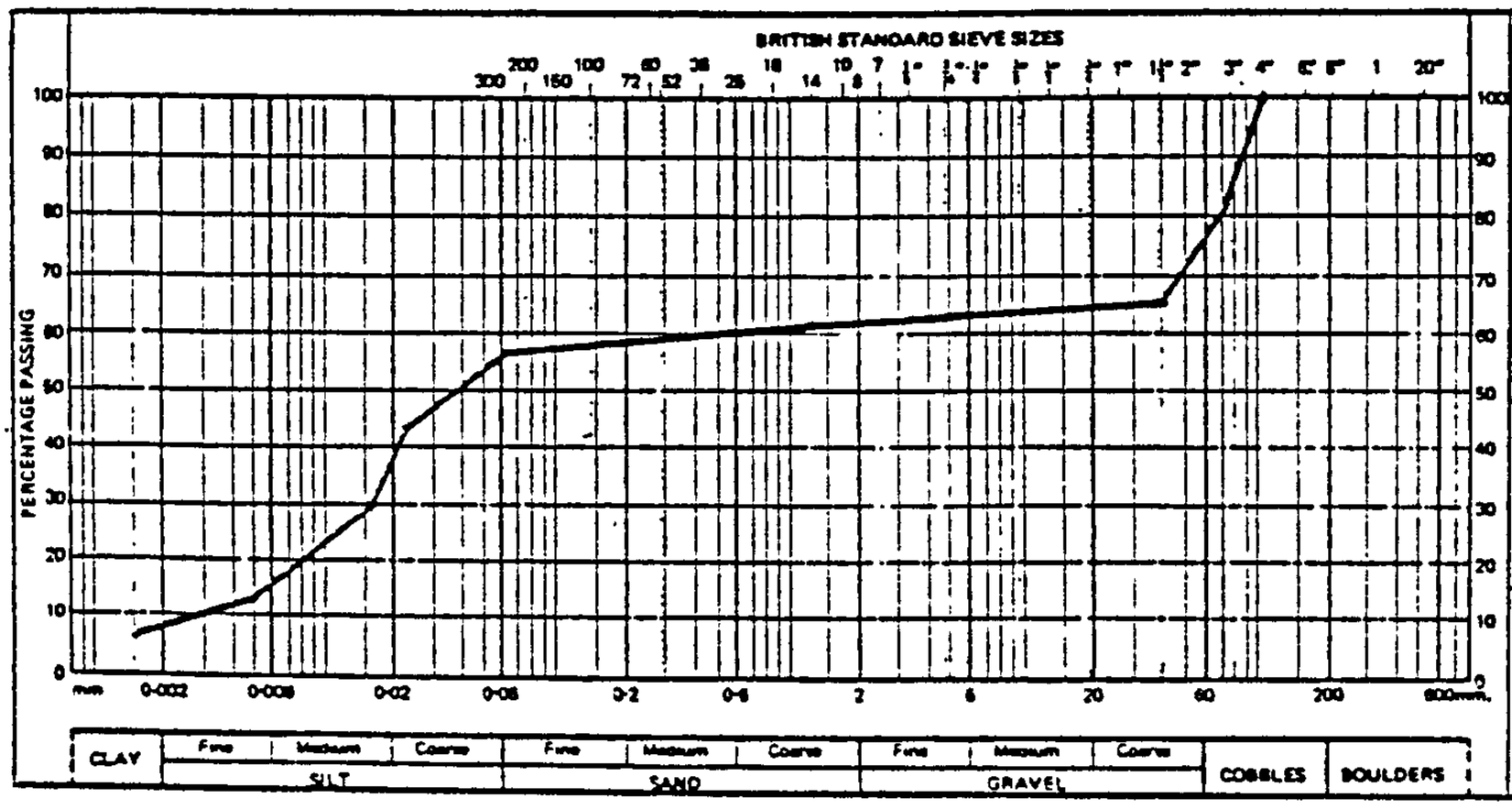
Sample No. 37

Depth: 20.5 m

Remarks

100 mm tube sample

silty SAND



(c)

Borehole No. 8

Sample No. 46

Depth 25.2 m

Remarks

bulk sample

gravelly SILT

Traced | Approved

Tested | Checked

Particle Size Distribution
GRADING CURVES

Used to show the results of Grading (gravel/sand) and Sedimentation (silt/clay) Tests

L87 3/74

Soil Mechanics

National Coal Board
BLACKBOY, CO DURHAM

Loc. No. 7693/2

Fig. 10

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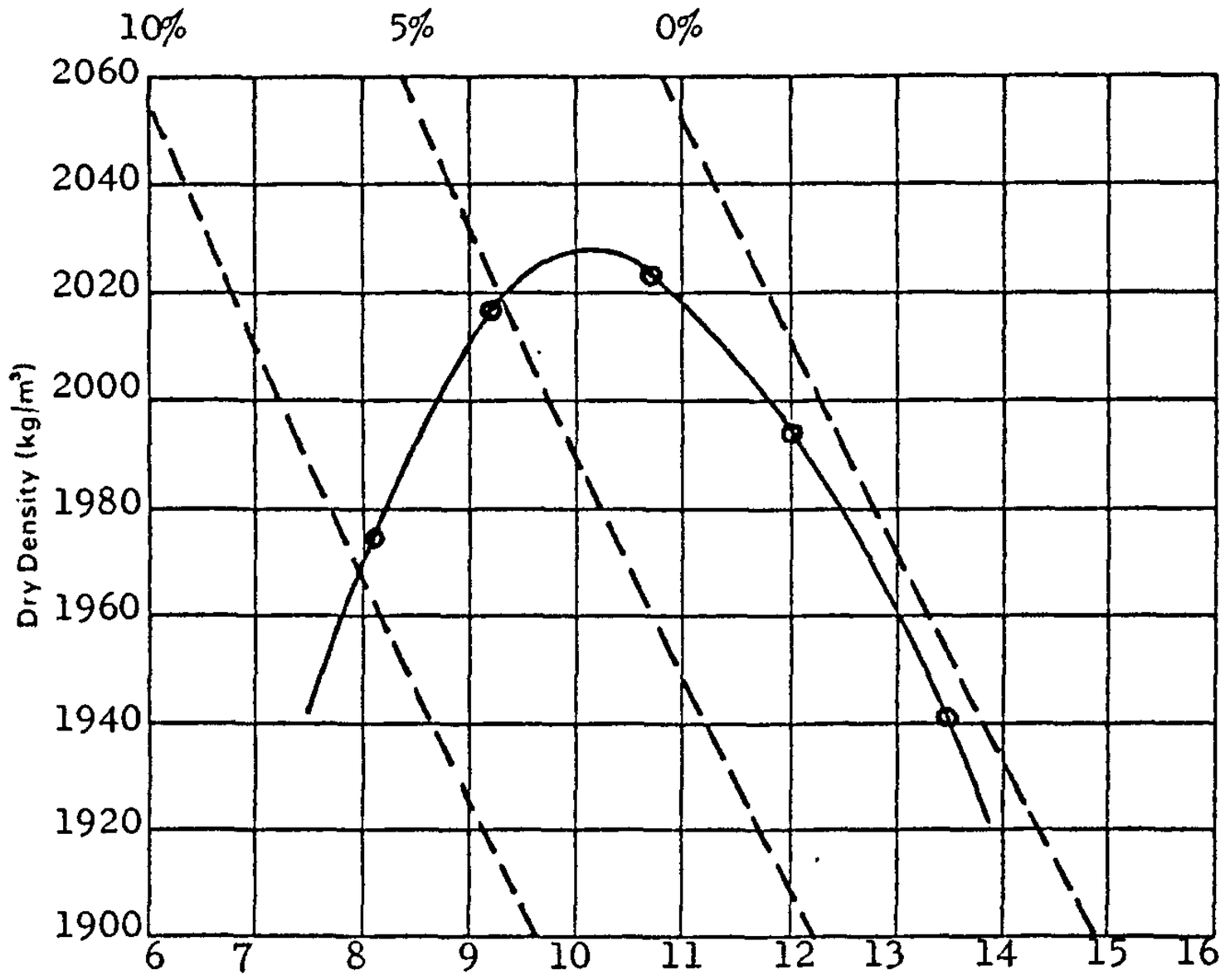
NDC Geotechnical Consultants

REPORT No. 006/N/666

FIGURE No. 11

B.S. COMPACTION TEST

BUTTERWELL DISPOSAL POINT



B.S. Heavy Compaction

Borehole No. B108

Sample Level 1.00–4.00m BGL

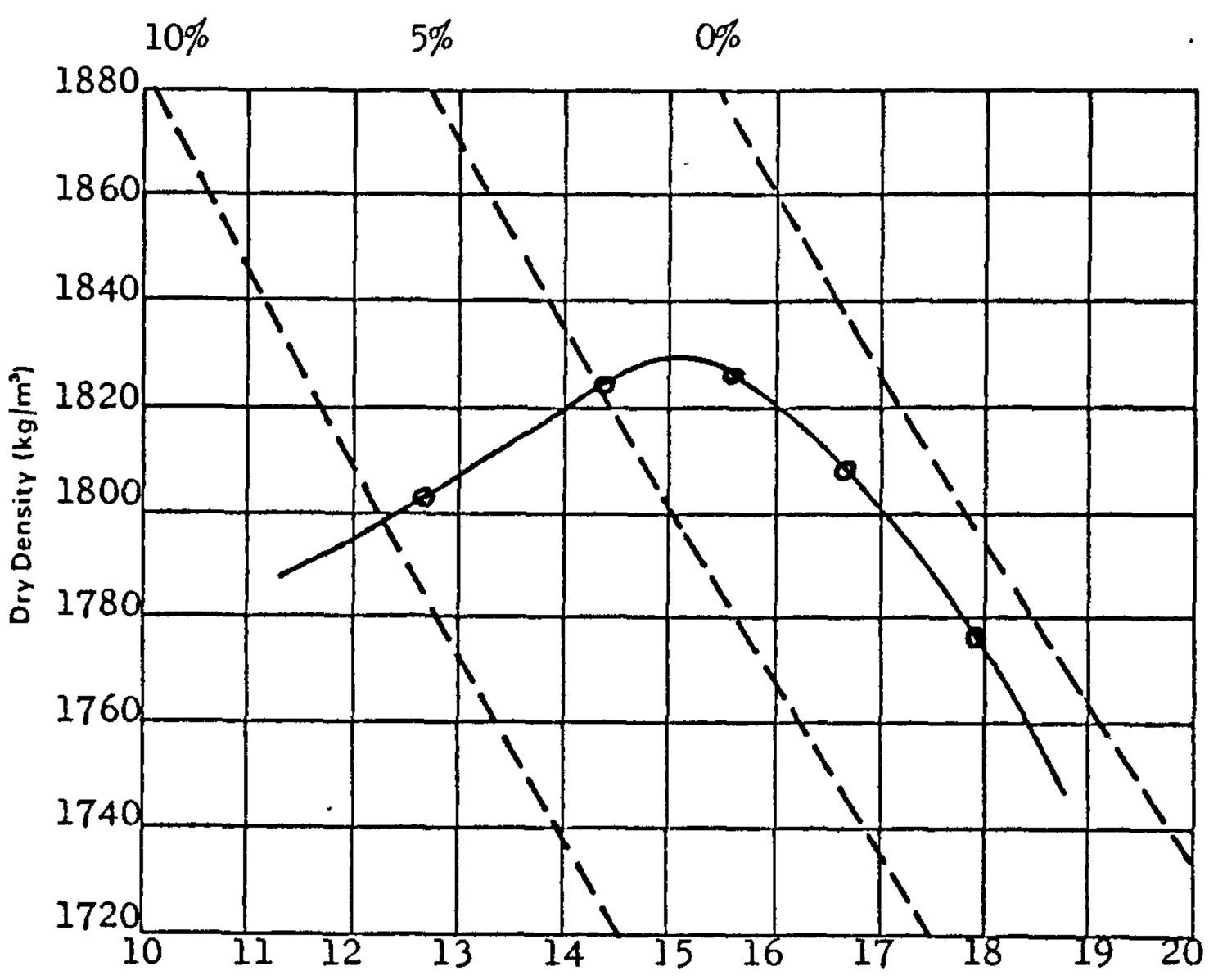
Sample Description

See Table 7 for descriptions.

Maximum Dry Density 2025 kg/m

Optimum moisture content 10.0%

Specific Gravity 2.65 assumed



B.S. Light Compaction

Borehole No. B108

Sample Level 1.00–4.00m BGL

Sample Description

See Table 7 for descriptions.

Maximum Dry Density 1830 kg/m³

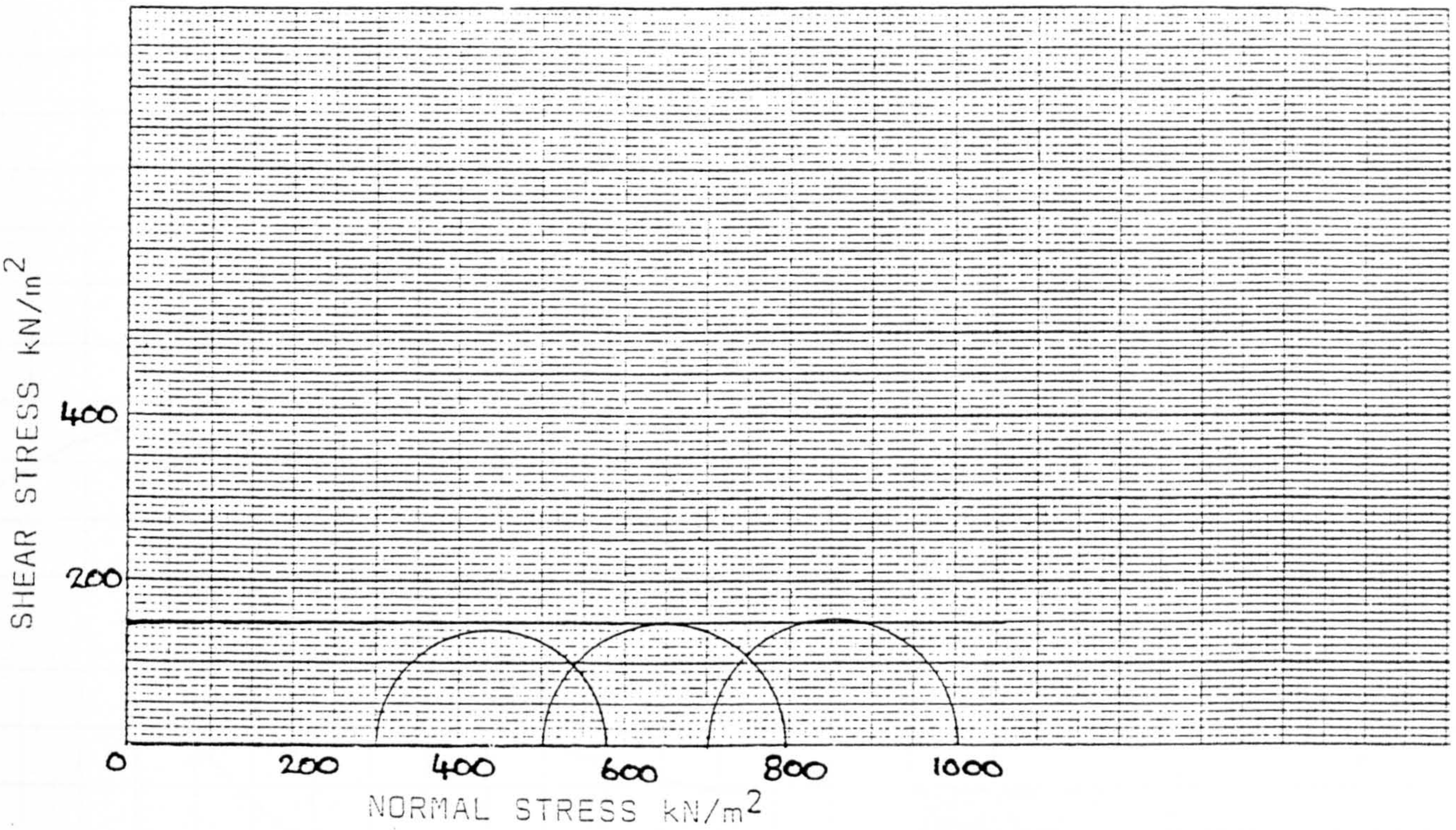
Optimum moisture content 15.0%

Specific Gravity 2.65 assumed

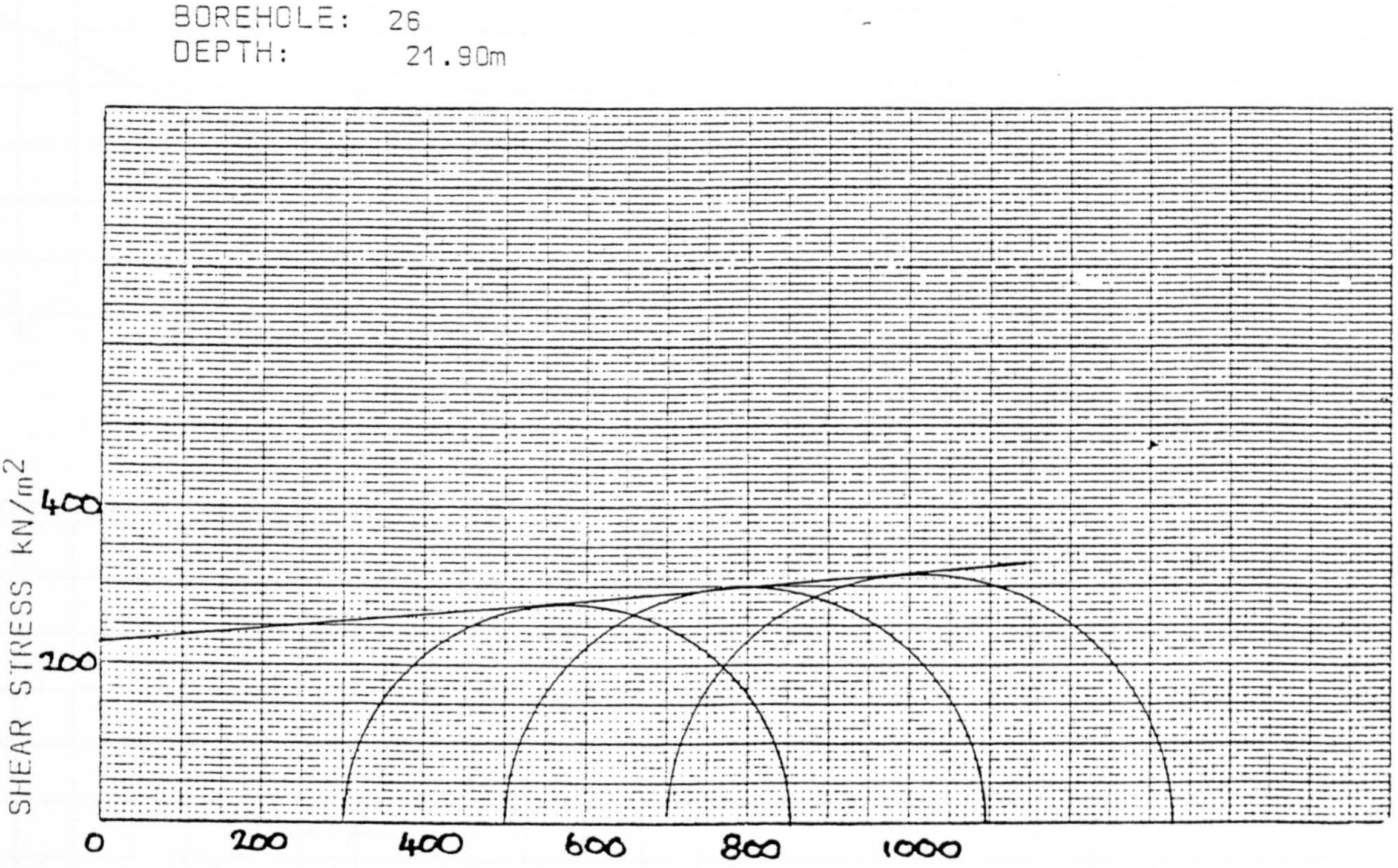
Used to investigate the moisture content/dry density relationship for a soil which is going to be used as fill.

FIGURE 12

QUICK UNDRAINED TRIAXIAL COMPRESSION TEST



BOREHOLE: 26
DEPTH: 21.90m



BOREHOLE: 26
DEPTH: 25.50m
LOCATION: Rose Hills O.C.C.S.

Norwest Holst Soil Engineering Limited

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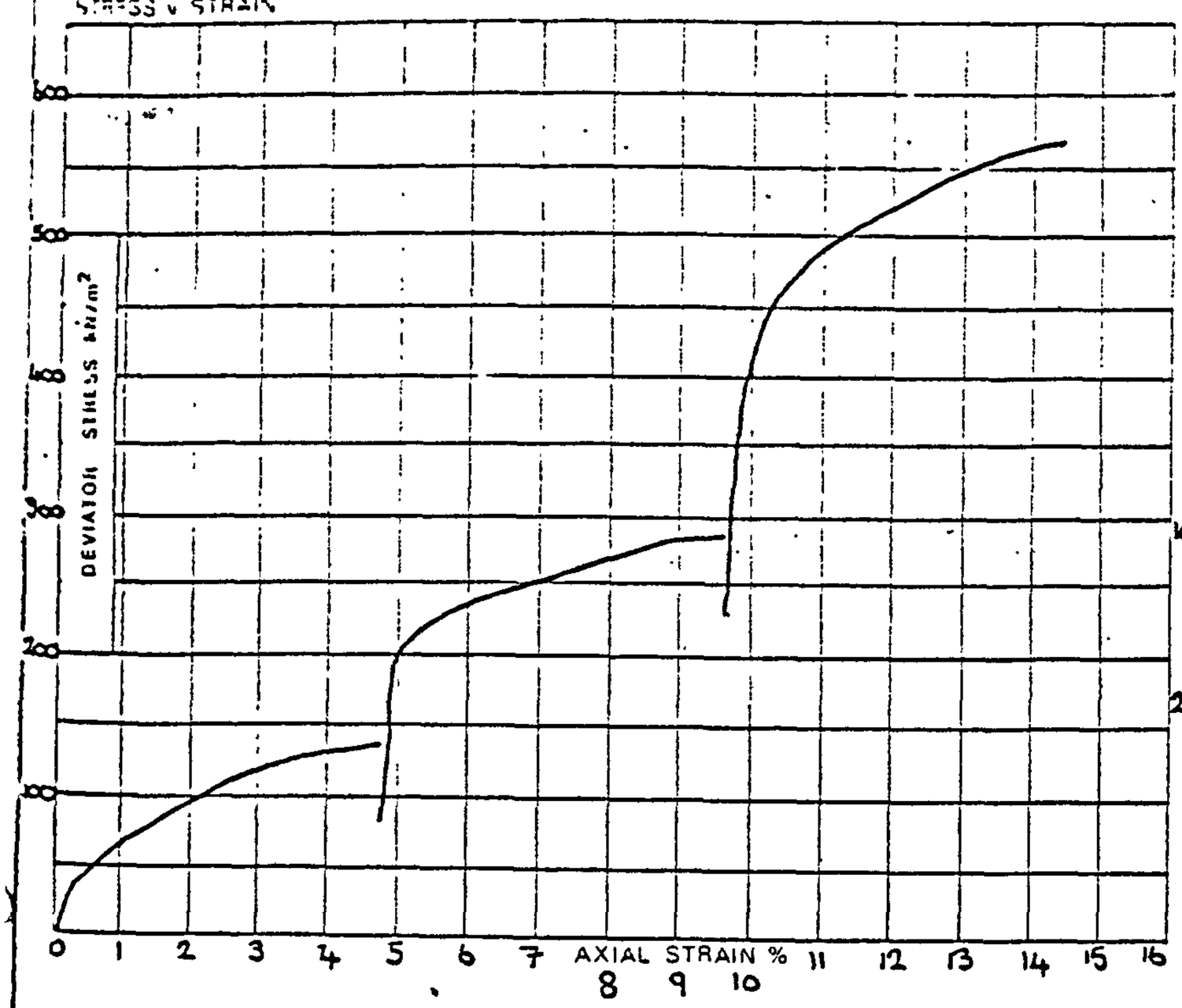
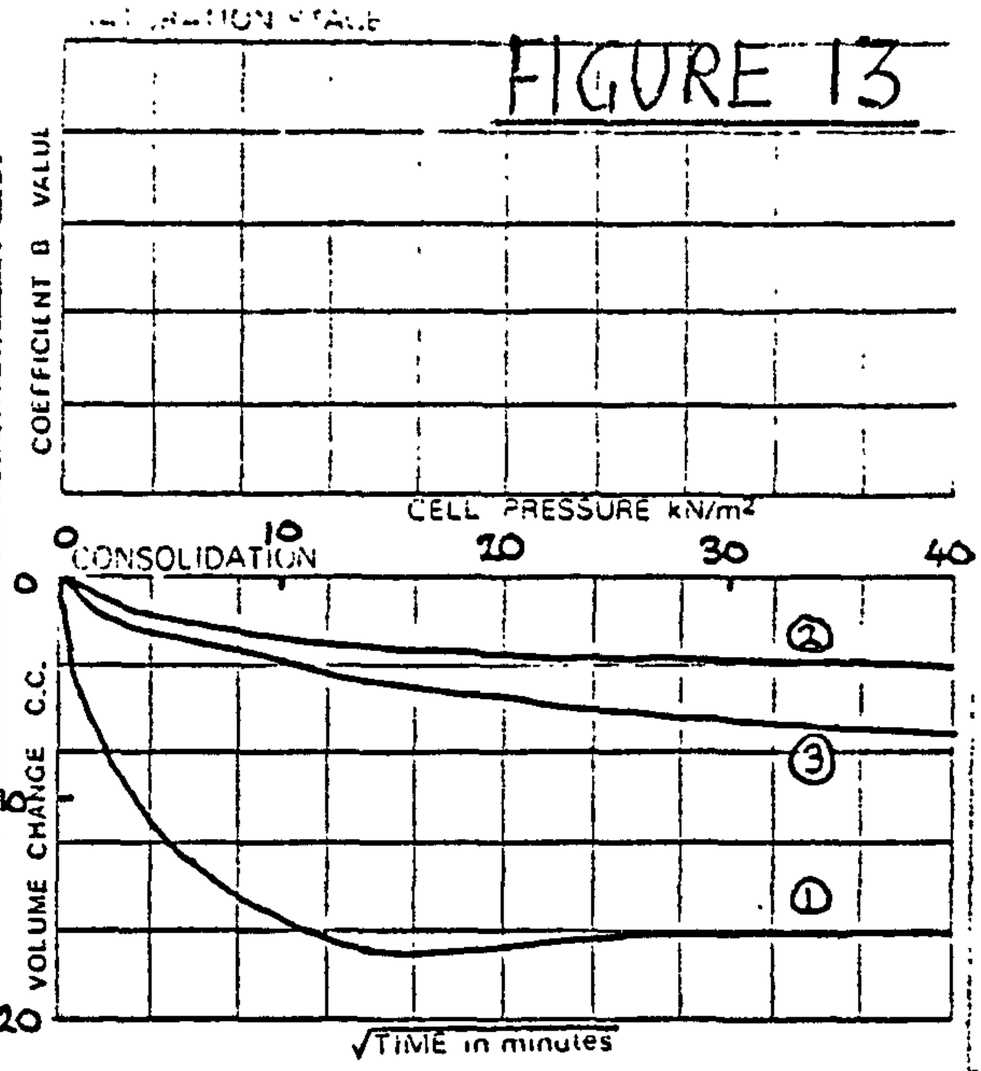
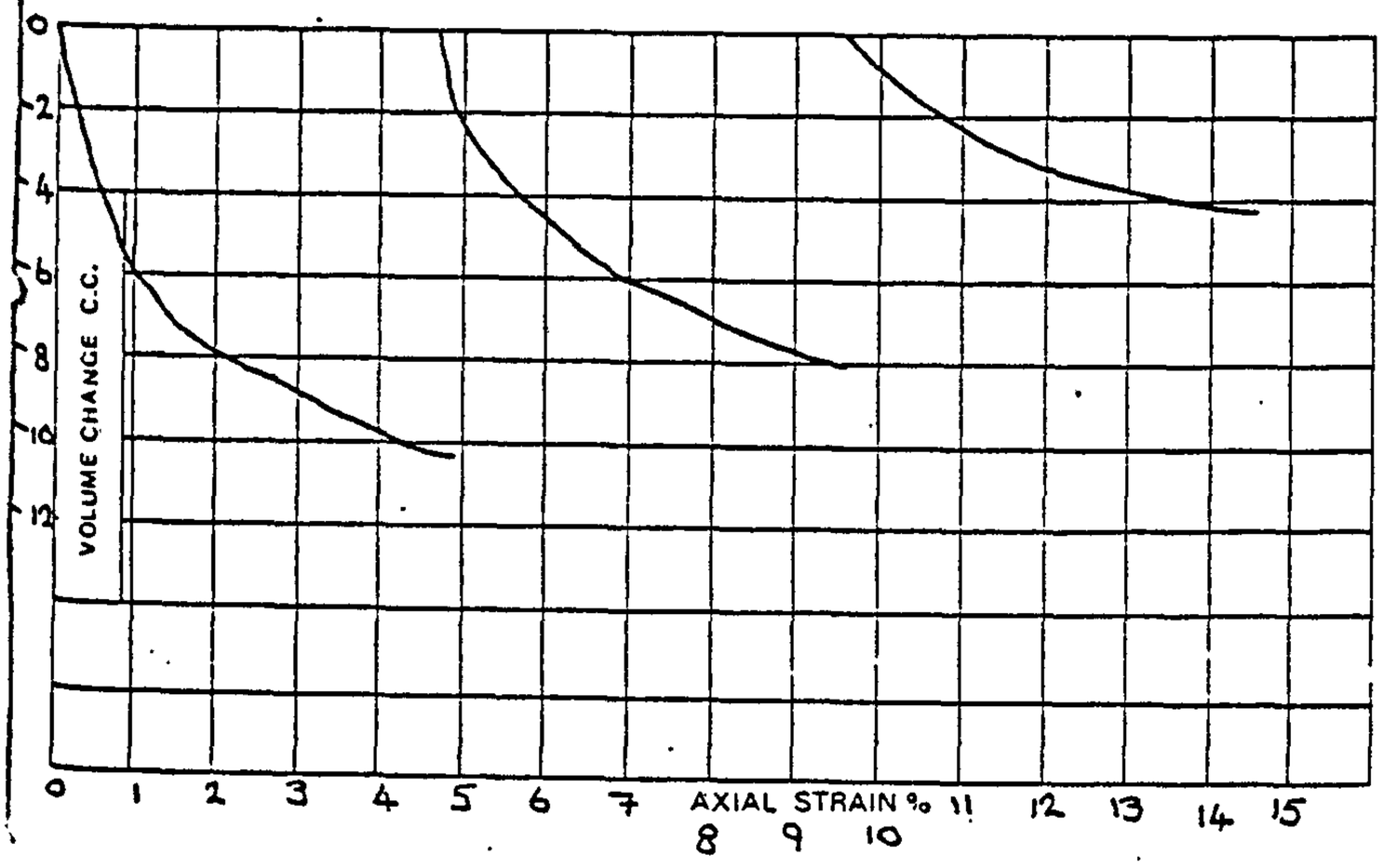
SITE Rose Hills O.C.C.S... BH No. 9... DEPTH 5·90m...

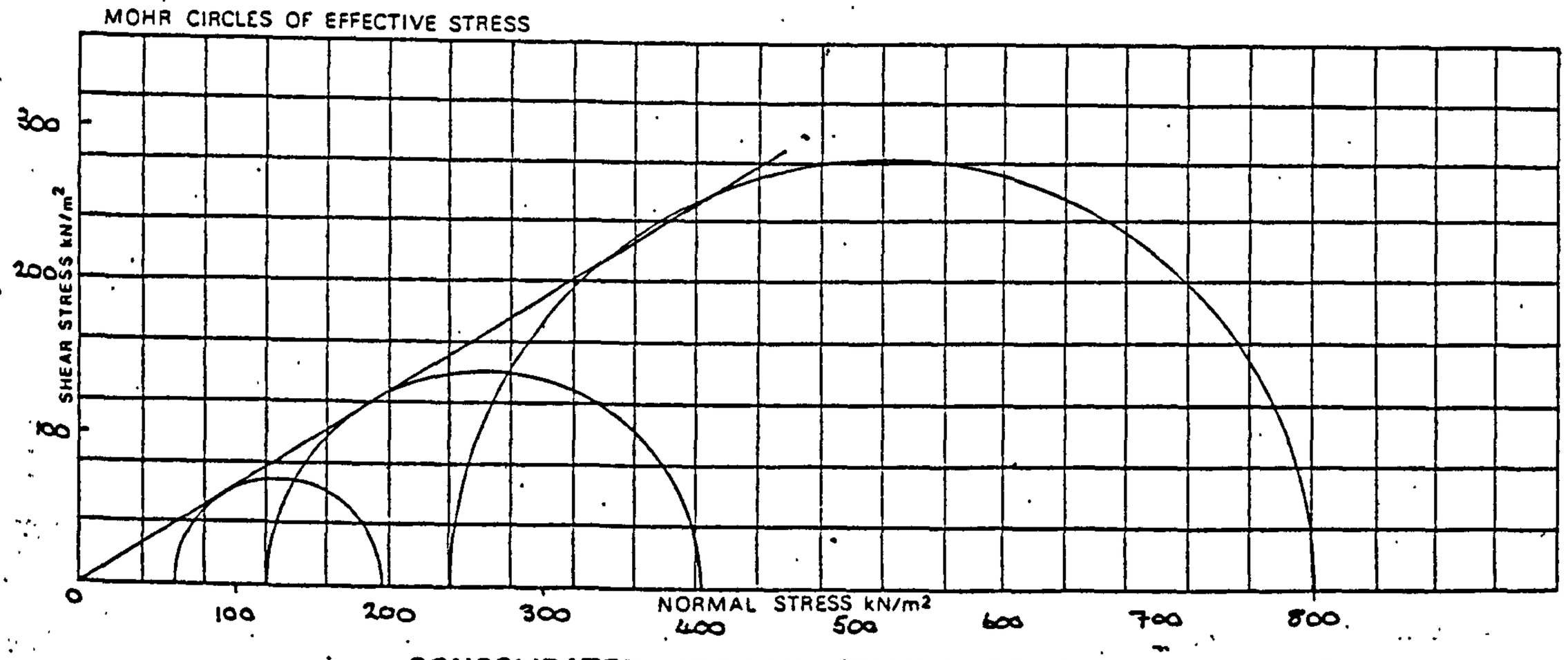
FIGURE 13







SPECIMEN NUMBER		1	2	3
EFFECTIVE CELL PRESSURE kN/m²		60	120	240
BACK PRESSURE kN/m²		—	—	—
PORE PRESSURE PARAMETER B		—	—	—
BULK DENSITY Mg/m³	Initial	2·20	2·21	2·21
	After Consolidation	2·21	2·21	2·21
MOISTURE CONTENT %	Initial	10·7	10·3	10·1
	After Consolidation	10·3	10·2	10·0
COEFFICIENT OF PERMEABILITY	m/sec			
MACHINE RATE OF STRAIN	mm/min	0·004		
DEVIATOR STRESS AT FAILURE	kN/m²	136*	286*	564*
MEMBRANE CORRECTION	kN/m²	—	—	—
FILTER DRAIN CORRECTION	kN/m²	—	—	—
EFFECTIVE SHEAR STRENGTH PARAMETERS	C' kN/m²		0	
	Ø' degrees		32	
SAMPLE DIMENSIONS		102mm dia. x 203mm		
DESCRIPTION		See borehole log		
NOTE		* Includes membrane and filter drain corrections		



CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST

Norwest Holst Soil Engineering Limited

FIGURE 14

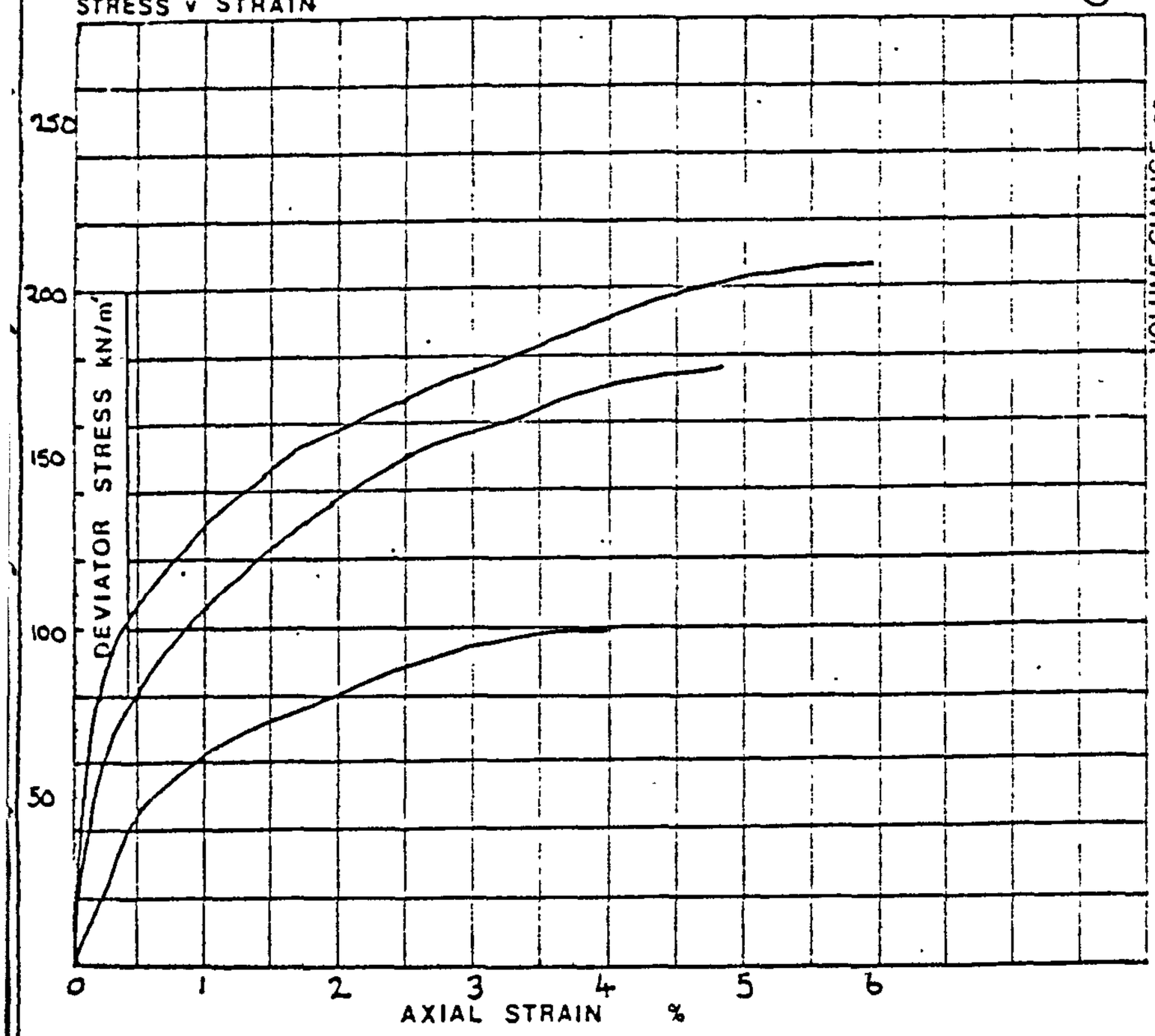
Site: Rose Hills o.c.c.s

Samples tested: ③ ② ①

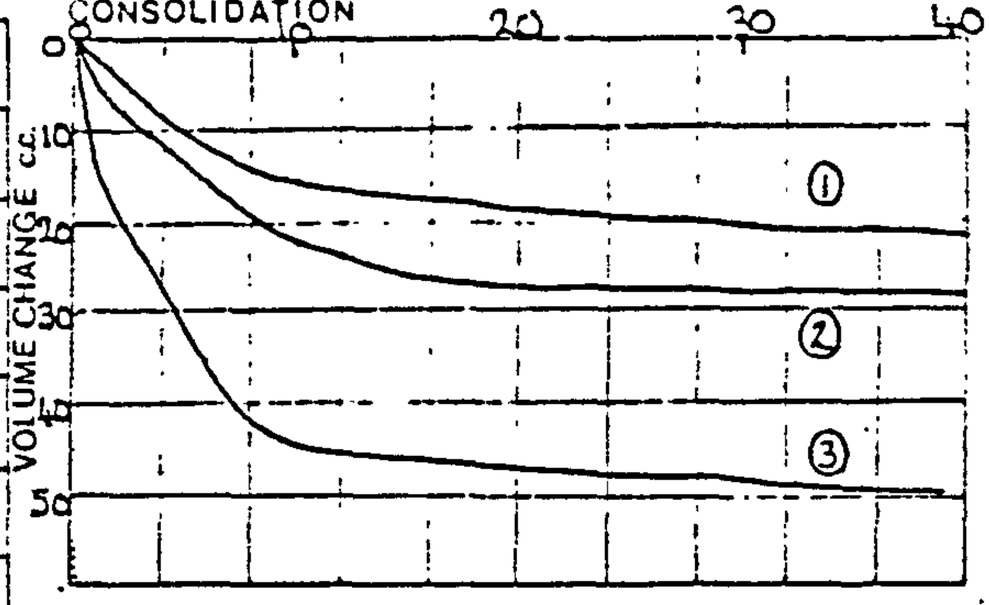
12 u 2·50m
14 u 2·00m
20 u 1·00m

Borehole No: ___
Depth ___

STRESS v STRAIN

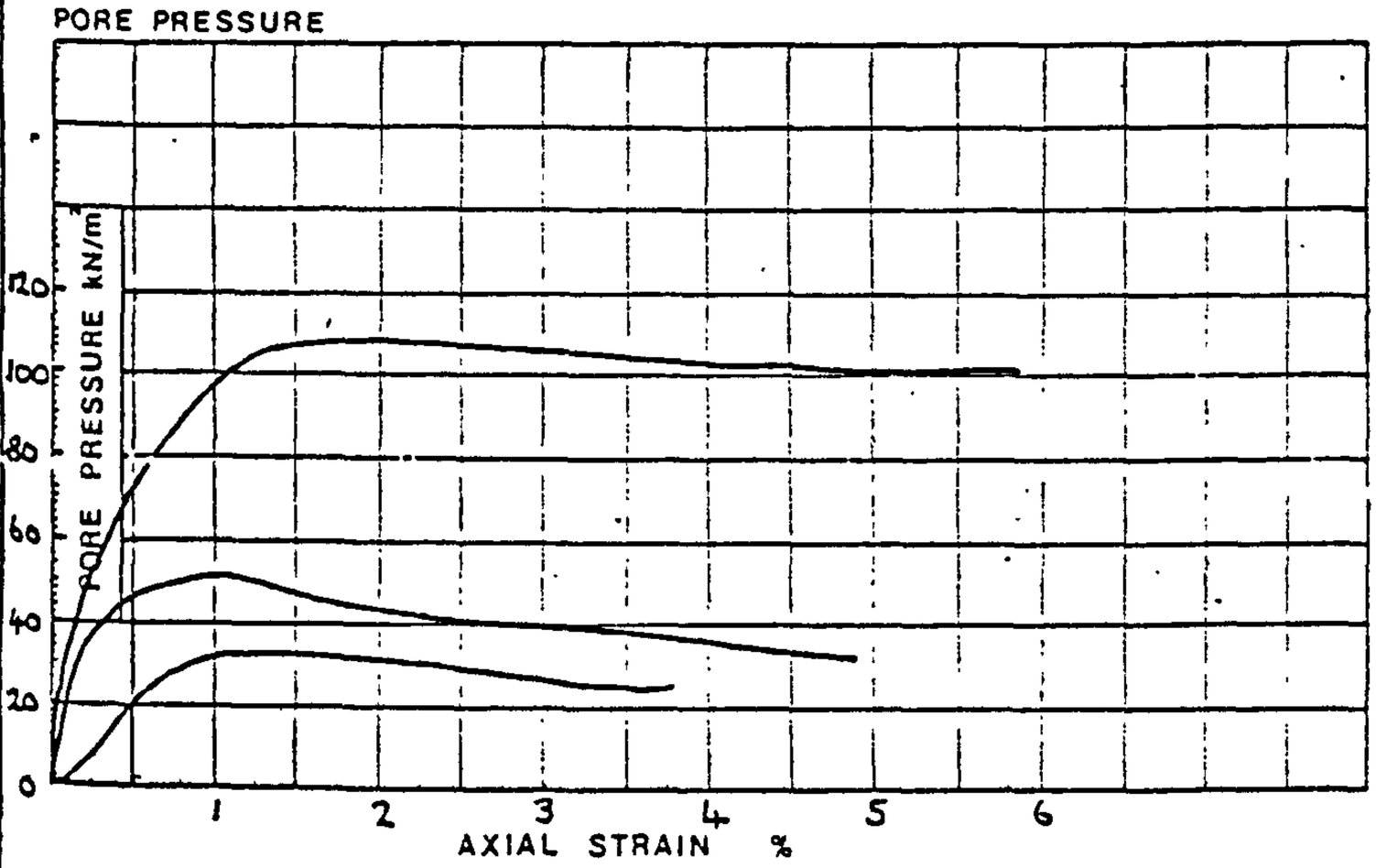


CONSOLIDATION



√TIME (mins)

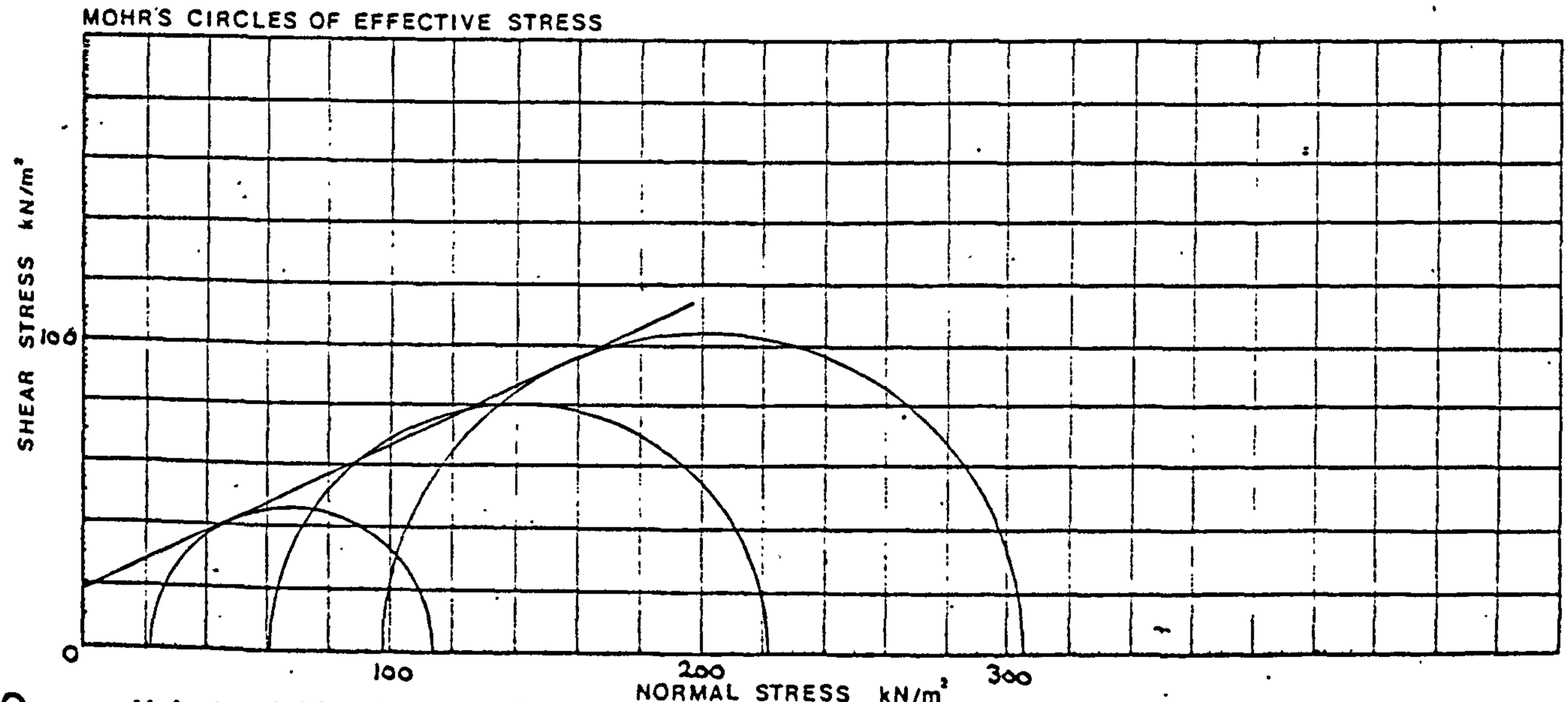
PORE PRESSURE



SPECIMEN NUMBER		1	2	3
EFFECTIVE CELL PRESSURE kN/m²		50	100	200
BACK PRESSURE kN/m²		190	140	140
PORE PRESSURE PARAMETER B		0·99	0·99	0·99
BULK DENSITY Mg/m³	Initial	2·15	2·16	2·10
	After Consolidation	2·18	2·19	2·12
MOISTURE CONTENT %	Initial	17·6	14·3	16·3
	After Consolidation	18·0	14·8	15·3
COEFFICIENT OF PERMEABILITY	m/sec			
MACHINE RATE OF STRAIN	mm/min	0·006		
DEVIATOR STRESS AT FAILURE	kN/m²	91 *	160 *	206 *
EFFECTIVE SHEAR STRENGTH PARAMETERS	C' kN/m²	18		
	Ø' degrees	26		
TYPE OF SPECIMEN		102mm.dia. X 203 mm.		
DESCRIPTION OF SOIL		See borehole logs		

NOTES: * Includes membrane and filter drain corrections.

MOHR'S CIRCLES OF EFFECTIVE STRESS



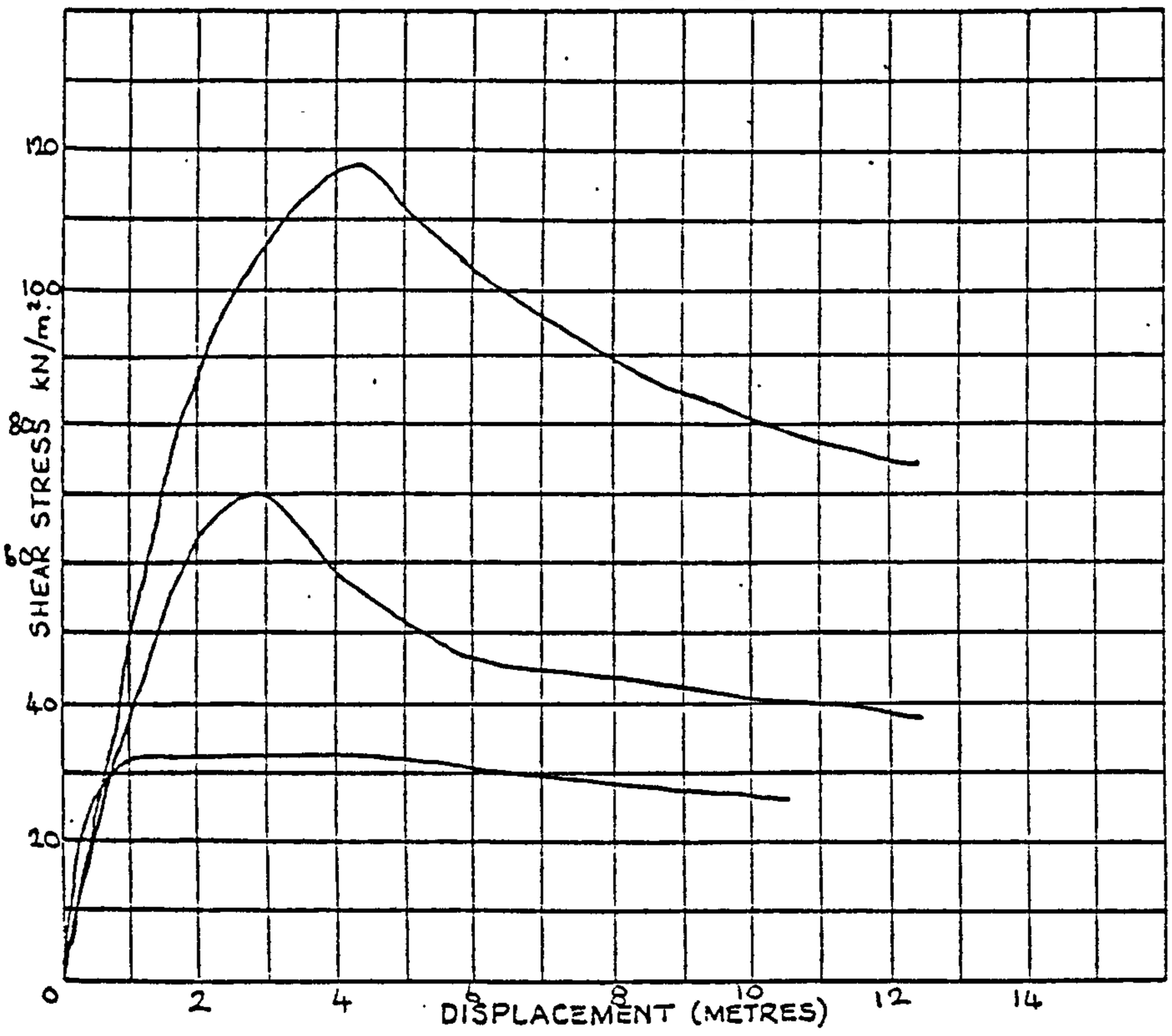
Consolidated Undrained Triaxial Compression Test with P.W.P. Measurement

TEXT CUT OFF IN ORIGINAL

SITE ACKLINGTON ..... BOREHOLE No. 2 ..... DEPTH 6:45m .....

FIGURE 15

SHEAR STRESS v DISPLACEMENT

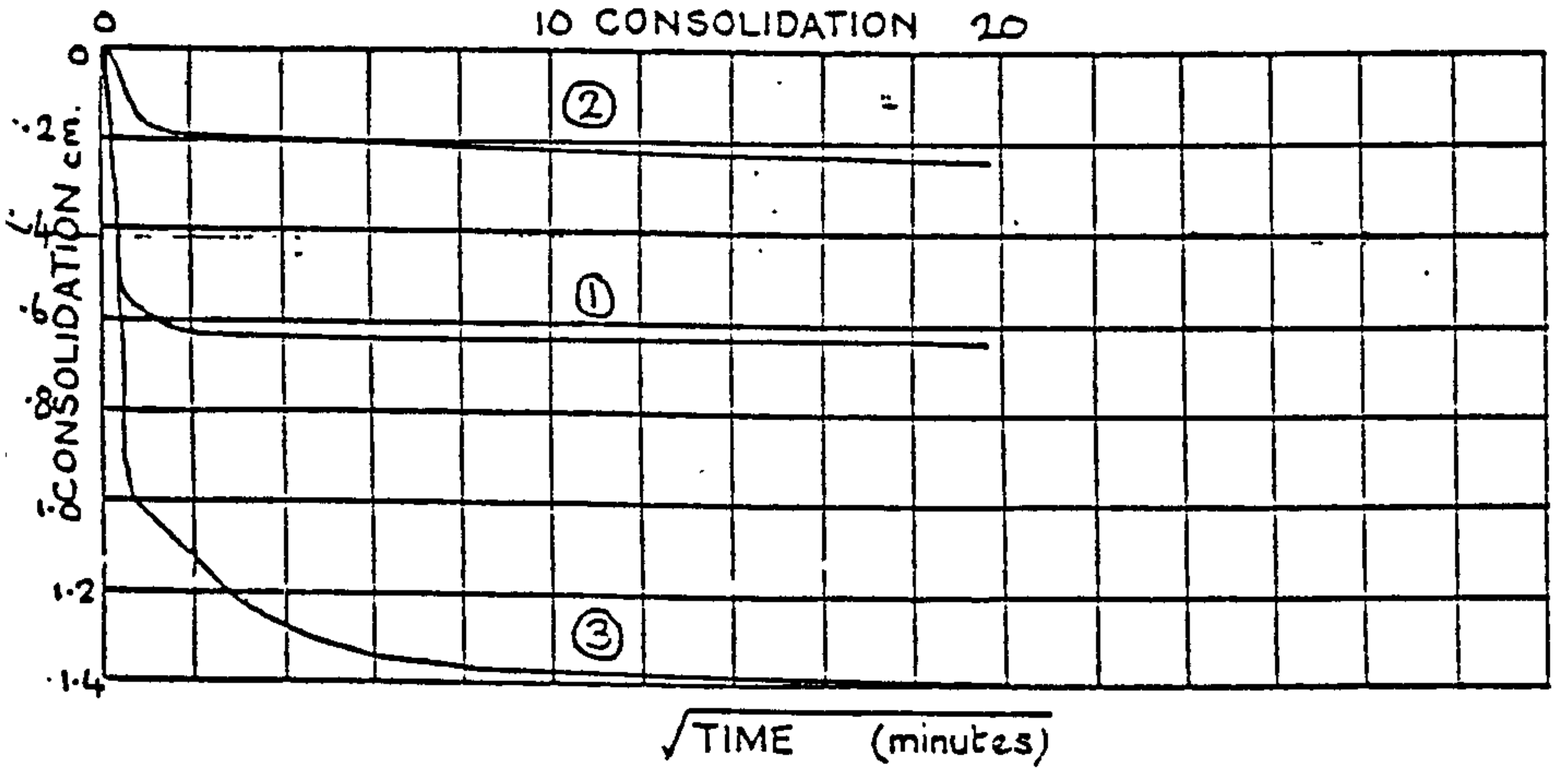


SPECIMEN NUMBER		1	2	3
VERTICAL PRESSURE kN/m²		67	134	268
PEAK STRENGTH kN/m²		32	70	118
STRAIN %		1·1	2·9	4·4
RESIDUAL STRENGTH kN/m²		—	—	—
STRAIN %		—	—	—
VOIDS RATIO	Initial	—	—	—
	Final	—	—	—
MOISTURE CONTENT %	Initial	31·7	30·1	30·6
	Final	33·4	30·0	29·5
DRY DENSITY Mg/m³	Initial	1·46	1·49	1·49
	Final	1·51	1·53	1·61
DIMENSIONS mm.	Initial	20·00	20·00	20·00
	After consol.n	19·37	19·54	18·56
	Final	19·38	19·45	18·40
AREA cm²		36	36	36
EFFECTIVE PEAK SHEAR STRENGTH PARAMETERS	C'p kN/m²	4		
	Ø'p degrees	22		
EFFECTIVE RESIDUAL STRENGTH PARAMETERS	C'r kN/m²	—		
	Ø'r degrees	—		

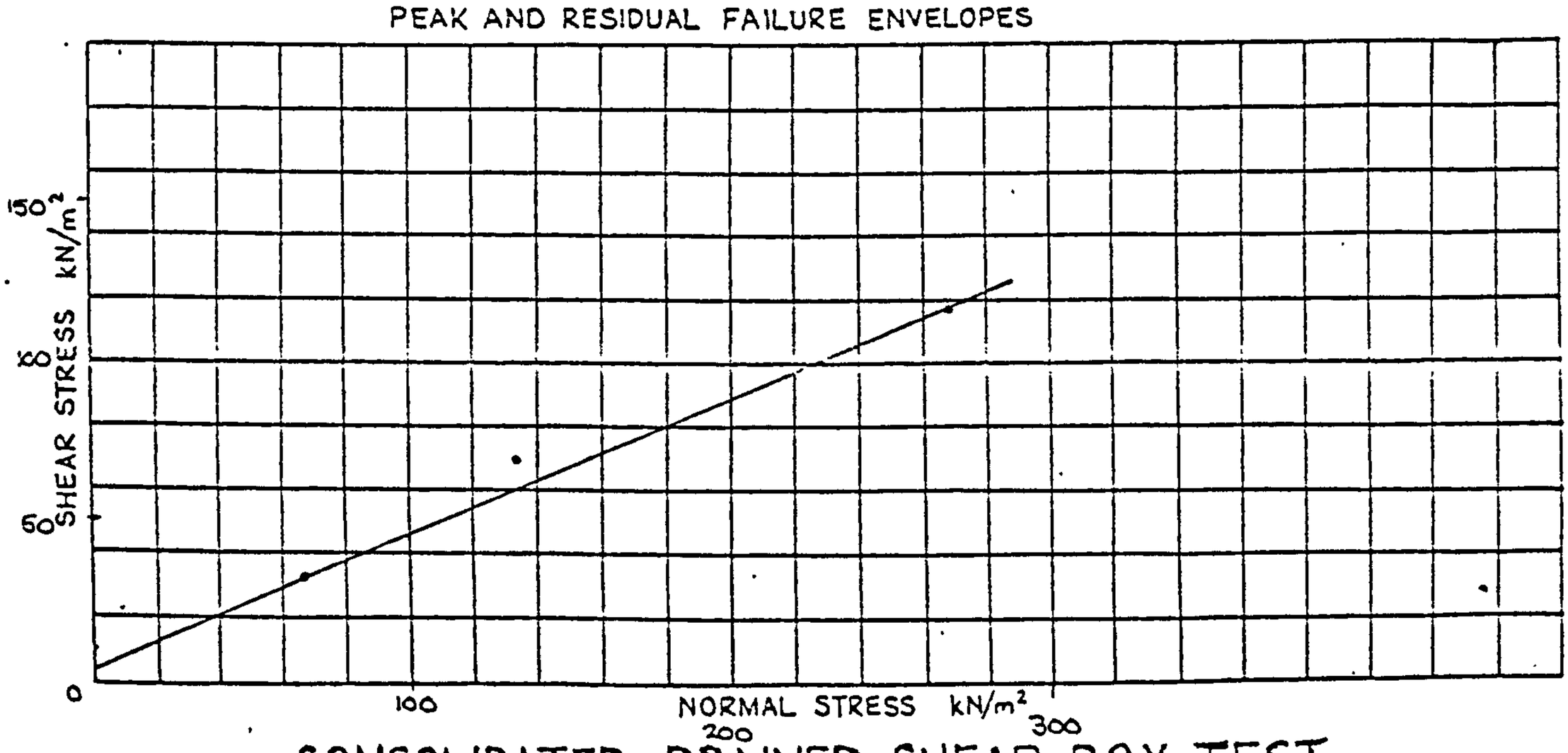
DESCRIPTION OF SOIL.

See borehole log

NOTES



PEAK AND RESIDUAL FAILURE ENVELOPES



CONSOLIDATED DRAINED SHEAR BOX TEST

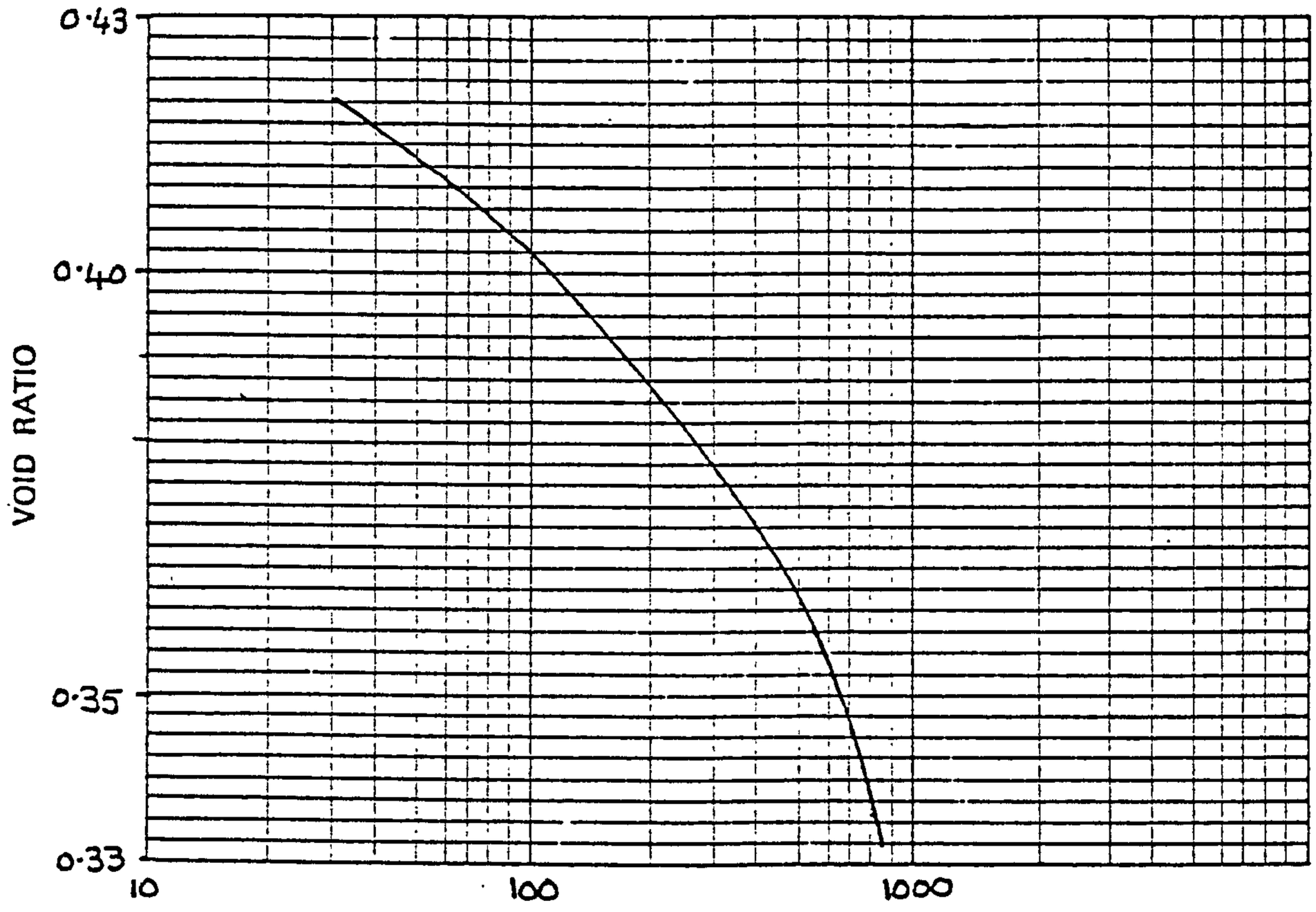
Norwest Holst Soil Engineering Ltd.

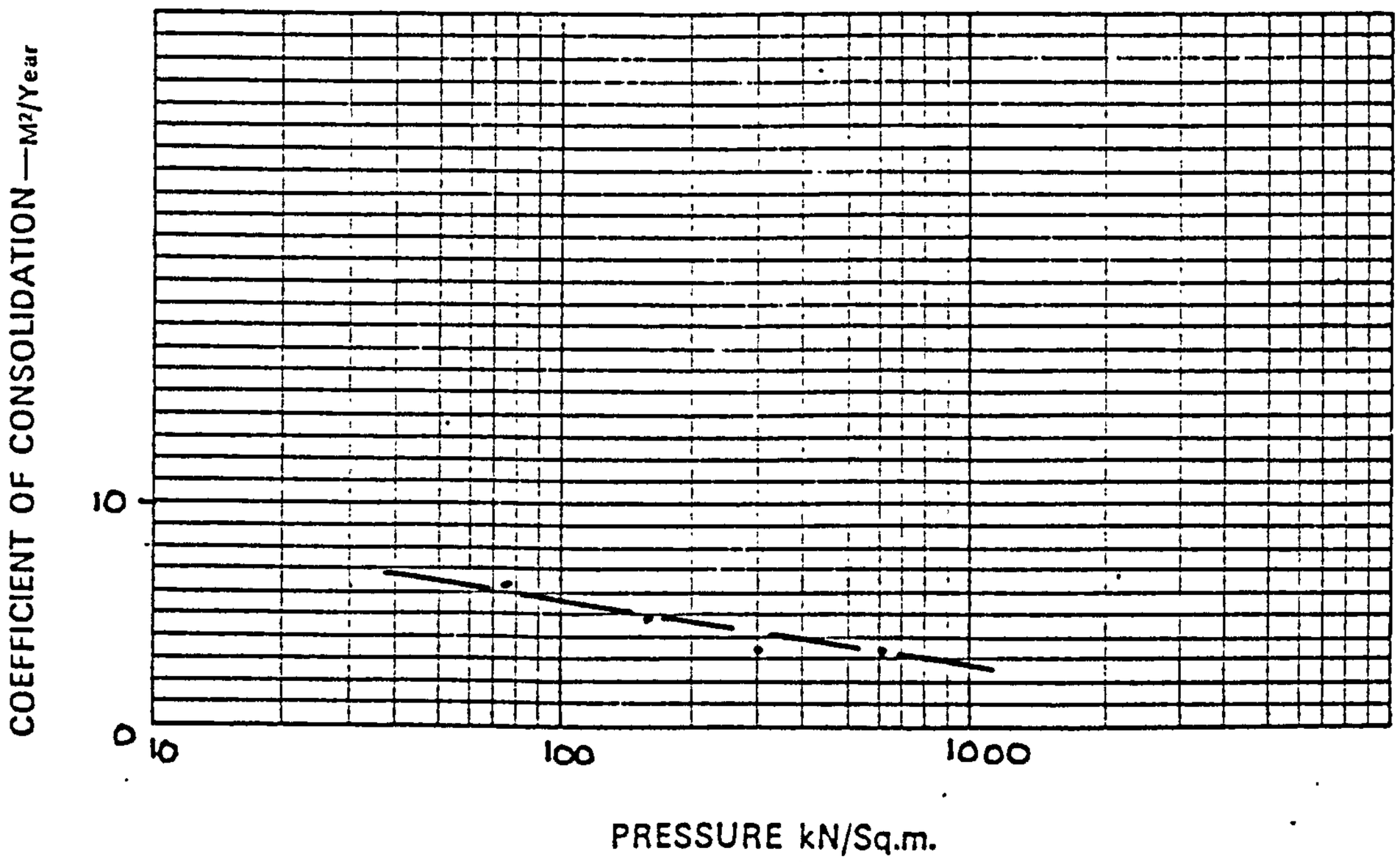
CONSOLIDATION TEST

FIGURE 16

CONTRACT No. F4736 LOCATION Rose Hills O.C.C.S.
DEPTH 7.00m BOREHOLE No. 5

INITIAL MOISTURE CONTENT 16.3 % FINAL MOISTURE CONTENT 14.0 %
INITIAL THICKNESS mm INITIAL DENSITY Mg/m³
INITIAL VOID RATIO FINAL VOID RATIO
SPECIFIC GRAVITY





5. APPLICATIONS

To cover all the applications of site investigation to engineering works associated with opencast mining would require a vast text book covering slope stability, foundations, carriageways, old workings, tunnelling, excavations and dewatering etc., etc.
Instead, it has been decided to illustrate the use of site investigations and soil mechanics parameters for some recent case studies from North East England.

5.1. West Linton O.C.C.S. (Proposed)
Stability of Batters in Superficials adjacent to B.R. (London-Edinburgh) Main Line

Figure 17 shows the main line railway running along the western side of the proposed coaling area (Area A). Between 6 and 10m of glacial drift overlies the easterly dipping Coal Measures bedrock. Within the predominantly boulder clay drift are extensive pockets or bands of water-bearing sand.

To check the long term safety of the railway, plus an overhead electricity line (which is to be routed between the railway and the Authorised Excavation Limit), a site investigation was carried out. This consisted of cable percussion boreholes on the section lines (A1-A2, A3-A4, A5-A6, A7-A8) as shown on Figure 17. Piezometers were also installed in some of the boreholes. From the borehole and laboratory testing information the strata succession, plus soil parameters and groundwater levels, were plotted for each section.

Figure 18 shows the strata succession, the density and effective shear strength parameters, and the groundwater details for Section A7-A8.

The stability of the proposed batters has been analysed using both the Bishop circular method and the Janbu non-circular method. The analyses were carried out using the N.C.B. 'SLOPEFOS' computer program, via Compower Ltd., at Cannock.

It can be seen that the overall Factor of Safety F for the superficials (Stratum 4, 5 and 6) is 1.868, which is quite safe. However, a very low F of 0.558 was obtained for the base of the sand layer (Stratum 5), this being due to seepage from the perched water table in the sand, and which represents minor slumping and backsapping effects that will occur when the sand is exposed.

A non-circular (Janbu) analysis for failure through the Coal Measures gave an adequate Factor of Safety F = 1.498.

Generally, in a situation like this one, where the long term safety of important property is being considered, we are looking for a Factor of Safety of at least 1.3. However, the seepage and slumping that will occur in the sand layer is predicted to be fairly minor and is not expected to cause a breach of the Authorised Excavation Limit (Orange Line).

FIGURE 17

WEST LINTON

'AREA A'

SECTIONS
A1-A2, A3-A4, A5-A6, A7-A8.
SCALE 1:2500

WEST LINTON AREA A SECTION A7 – A8 Slope No 6

Scale 1 / 200

STRATUM 1	γ = 21·1 kN/m^3	
	c′ = 100 kPa	
	ϕ' = 35°	
STRATUM 2 (Assuming Clay Bands within Coal Seam)	γ = 21·1 kN/m^3	
	c′ = 11 kPa	
	ϕ' = 16°	
STRATUM 3	γ = 21·1 kN/m^3	
	c′ = 30 kPa	
	ϕ' = 32°	
STRATUM 4	γ = 22·3 kN/m^3	
	c′ = 15 kPa	
	ϕ' = 24·5°	
STRATUM 5	γ = 19·4 kN/m^3	
	c′ = 0	
	ϕ' = 37°	
STRATUM 6	γ = 20·2 kN/m^3	
	c′ = 4 kPa	
	ϕ' = 23·5°	

1 on 2 Drift
5m Rockhead bench
1 on ½ Rock

(19,34) (35,34) F=1·868 (27,22) (35,22) (19,20) (35,20) F=0·558 (50,18) (38·5,17·8) (33·8,15·4) (35,15) (42,15·4) (27,15) (30,13·5) (34,14·5) (38,15) (29 13) (50,15·4) (50,12·5) (50,10·2) (17·5,9·7) (22·5,9·8) (37,10) (30,9) (22,7) (17,5) (35·5,6) (34,4·7) (50,6) (50,5) (14·5,3·1) (14,2) (0,1)

W.L. W L

Boulder Clay

Sand

Boulder Clay

Mudstone

Coal (G100)

Mudstone

Borehole A8

Borehole A7

Janbu backscar F=1·498

ORANGE LINE

STRATUM 6
STRATUM 5
STRATUM 4
STRATUM 3
STRATUM 2
STRATUM 1

123m A.S.D.

Y

X

0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50

34 32 30 28 26 24 22 20 18 16 14 12 10 8 6 4 2

FIGURE 18

5.2 Reclamation of Stobswood Pit Heap (Sisters O.C.C.S.)

The disused spoil heap for the former Stobswood Colliery was to be reclaimed as part of the restoration of Sisters O.C.C.S. During the early stages of the earthmoving operations on the spoil heap, old unrecorded slurry lagoon deposits were discovered and ground heave occurred in the perimeter drainage ditch adjacent to the B.R. (London - Edinburgh) main line.

Inspection of the site indicated that the ground heave or ditch displacement was a very local problem at the toe of the spoil heap. However, because of the proximity of the main line railway, and the possible serious consequences of any further heave occurring beyond the ditch and affecting the rail alignment, it was decided to carry out a full ground investigation and stability analysis.

Twelve cable percussion boreholes were sunk, both along the toe and through the crest of the partially reclaimed spoil mound.

Figure 19 shows a cross-section of the spoil heap perpendicular to the railway and parallel with the maximum slope gradients on the heap. The profiles of the original heap and the planned reclamation slope are shown, plus the intermediate stage when the ditch displacements occurred and the investigation was carried out.

Boreholes 2,4,5, 11 and 12 coincided with the section chosen for analysis (Figure 19). This section shows the strata succession and the groundwater levels. In the natural soils underlying the heap is a fairly continuous layer of laminated clay (Stratum 4), and a layer of silty sandy clay which was quite soft (Stratum 6) underlying the toe of the heap but not continuous under the main body of the heap. Within the heap, and coming right to the toe was a continuous layer of slurry lagoon material (Stratum 7).

From the borehole samples and subsequent laboratory tests, density and effective shear strength parameters were obtained. A Bishop type stability analysis was carried out for sets of failure circles passing through the laminated clay, and through the lagoon material and silty sandy clay, using the N.C.B. 'SLOPEFOS' program. These gave minimum Factors of Safety of 2.130 and 2.169 respectively, and of course show that the reclaimed spoil heap profile is abundantly safe.

It is thought that the ground movements in the perimeter drainage ditch were possibly brought about by earthmoving plant operating near the toe of the heap and thus causing the lagoon material and soft silty sandy clay to be displaced into the ditch.

STOBSWOOD PIT HEAP

STABILITY OF S.W. FINAL SLOPE AND RAILWAY EMBANKMENT
TYPICAL SECTION THRO' S.W. SLOPE LOOKING N.W.

FIGURE 19.

NUMBER	DESCRIPTION	PARAMETERS
STRATUM 9	Coarse Colliery Discard - Unburnt.	γ = 19·5 kN/m^3 c' = 0 ϕ' = 29°
STRATUM 8	Made Ground - Railway Embankment.	γ = 19·5 kN/m^3 c' = 0 ϕ' = 29°
STRATUM 7	Fine Colliery Discard - Slurry Lagoon Material.	γ = 12·7 kN/m^3 c' = 0 ϕ' = 33°
STRATUM 6	Silty Sandy Clay.	γ = 20·2 kN/m^3 c' = 5 kPa ϕ' = 18°
STRATUM 5	Sand and Gravel.	γ = 20·5 kN/m^3 c' = 0 ϕ' = 30°
STRATUM 4	Laminated Clay.	γ = 20·5 kN/m^3 c' = 10 kPa ϕ' = 18°
STRATUM 3	Sand and Gravel.	γ = 20·5 kN/m^3 c' = 0 ϕ' = 33°
STRATUM 2	Boulder Clay.	γ = 21·8 kN/m^3 c' = 2 kPa. ϕ' = 28°
STRATUM 1	Bedrock - Mudstone.	γ = 21·1 kN/m c' = 100 kPa ϕ' = 35°

LEVELS (Y CO-ORDINATES) IN METRES A.O.D.

Bishop analysis of circular slip surfaces using a grid of centres and circles touching a specific line.

CENTRE GRID DATA						No. OF
Xmin.	Xinc.	Xmax.	Ymin.	Yinc.	Ymax.	CENTRES
10	5	70	60	5	100	117

LINE CO-ORDINATES						
X	0	60	80	95	120	i.e. to base of stratum 6/7
Y	43·5	43·5	44	44	44	
X	0	30	60	95	120	i.e. to base of stratum 4
Y	40	40	38·5	38·5	38·5	

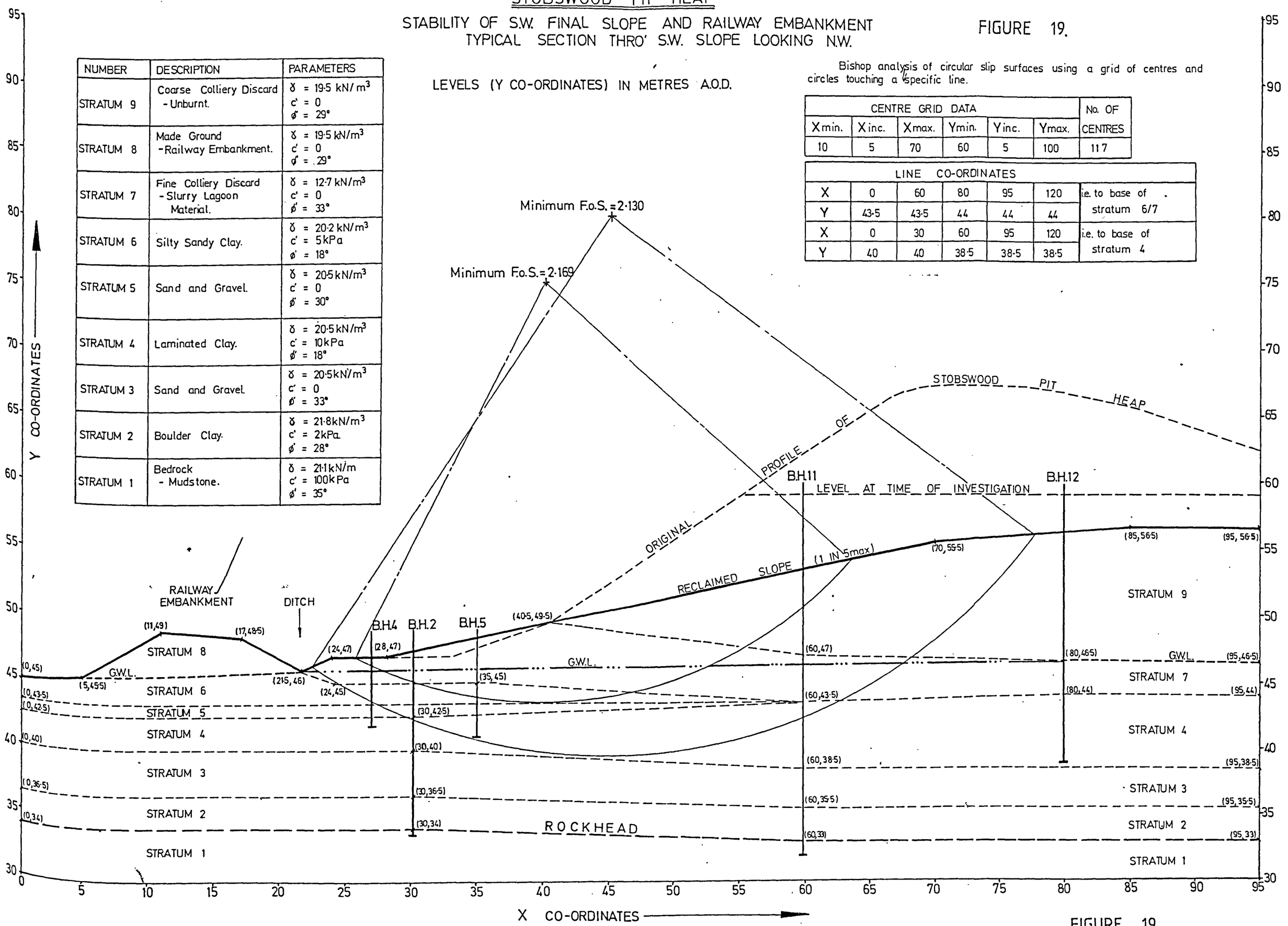


FIGURE 19.

5.3 Swalwell Coal Disposal Point

Two new structures have recently been completed at Swalwell D.P., being a new stocking out facility and a new smalls coal facility. The ground investigations carried out for these structures illustrate the four stages of site investigation as described in Section 2 of these notes.

The first stage included the seeking of information from previous investigations which had been carried out in the general vicinity of Swalwell D.P.. An approach was made to the Institute of Geological Sciences to see if they had details of any previous boreholes and this enquiry produced a lot of information, mainly from the site investigation carried out in 1967 for the now completed Gateshead Western By-Pass. The borehole locations are shown in Figure 20 and the ground conditions proved in these boreholes are summarised in Table 1. This table also includes a brief comparison of possible foundation types for the new structures, related to the strata succession.

It can be seen that the ground conditions could present difficult foundation problems, i.e. either the structures must tolerate large time dependent settlements or expensive piled foundations must be adopted. Also none of the I.G.S. supplied borehole locations are closer than 80m to either of the new structures.

5.3.1 FOUNDATIONS FOR NEW STOCKING OUT FACILITY

It was proposed to construct a new stocking out facility at the location shown in Figures 20 and 21. Brief details of the new construction are shown in Figures 22 and 23, i.e. the reinforced concrete structure comprises a rectangular base slab (approx. 20m x 30m) with retaining walls on three sides.

The second stage or main site investigation, comprising three cable percussion boreholes, was carried out at the site of this new structure and proved similar ground conditions to the I.G.S. supplied borehole information, Laboratory tests were carried out to determine total shear strength parameters, consolidation parameters and chemical characteristics (SO_3 and pH values). These ground conditions are shown in Figure 24, along with maximum shear stress distributions (after Jürgenson, 1934) and vertical pressure distributions for the centre and edges of the base slab (after Boussinesq, 1885). This investigation proved that the base slab founded 1m below the surface (i.e. in the made ground) would be adequate and that piles should not be needed.

Bearing capacity and settlement calculations for the base slab were carried out and the results are summarised below:-

(i) BEARING CAPACITY FAILURE:-

Minimum Factor of Safety (based on mean shear cohesion $c_u = 24.5kN/m^2$) = 2.74

(C.P. 2004 Foundations requires Min. F.o.S. of between 2 and 3)

(ii) SETTLEMENT:-

Maximum at western edge and centre of slab	= 280mm
Maximum at eastern edge of slab	= 190mm
Maximum at NE and SE corners of slab	= 90mm
Time for 90% settlement in mottled clay and organic clay	= approx. 5 years
Time for 90% setlement in laminated clay	= 12 years

The base slab will be subject to a loading and unloading cycle, but it is anticipated that the majority of the settlement will have occurred during the first 5 - 10 years.

(iii) CONCRETE:-

High levels of soluble sulphates (SO_3) plus a high water table were recorded in the made ground, hence sulphate resisting cement was incorporated into the base slab concrete.

On completion of the structure it was decided to monitor the settlement of the base slab to see how predicted and actual displacements compare (i.e. an "After Construction" or fourth stage investigation). This consists of a simple levelling check carried out monthly and the results obtained to date are shown in Figure 25. Because monitoring did not commence until three months after the structure was first fully loaded a correction has been applied to the measured settlements. The Figure shows that, so far, actual settlements are reasonably close to the predicted curves. Also the predicted settlement curves give an indication of the very large differential settlement that the structure has to withstand.

5.3.2 FOUNDATIONS FOR NEW SMALLS COAL FACILITIES ON 'A' PLANT

The location of this structure is shown on Figures 20 and 21. Brief details of the construction and function are shown in Figure 26 i.e. a steel structure about 13m high supported on four stanchions and housing a crusher and surge bunker.

A single cable percussion borehole was sunk at this location, and proved the ground conditions as shown on Figure 27 (it can be seen that the ground conditions are very similar to those proved below the new stocking out facility, and as shown on Figure 24).

The maximum loading from each stanchion was calculated to be 442 kN. With this intensity of loading and the tall slender type of structure it was apparent that shallow foundations would not be feasible (i.e. individual pad foundations would be liable to bearing failure; also a slab or raft would be liable to excessive total and differential settlements which could tilt the structure and/or disrupt the rail track that passes underneath).

The Company responsible for the design and construction of the structure chose to specify driven piles of precast concrete construction. 2 No. piles were installed under each stanchion foundation, giving 8 No. piles in total, the average pile length being 11m. The piles were designed to be end bearing in the dense sand and gravel stratum, and based on the 'N' values obtained in this stratum the calculated Factor of Safety (FoS) for a single pile was between 2.3 and 4.7 (depending upon the method of calculation used). The required set was calculated as being 20 blows per 44mm for a dynamic FoS of 2.5. Based on the strength of the underlying laminated clay, a FoS of 13.4

was obtained for the base of the pile group. The total settlement of the pile group was calculated to be 27mm and should be 90% complete within 25 years, (actually it is expected that the rate of settlement will be rather quicker than this). The sets achieved on driving all the piles were better than the value specified and ranged between 29 and 40 mm for 20 blows.

In order to verify the adequacy of the pile design it is usual to carry out a pile loading test. This amounts to a Foundation Investigation (or third stage investigation) as described earlier in these notes. Where a small group of piles is involved it is usual to select one of the working piles and to apply a proof load test as described in CP 2004 : 1972 - Foundations.

The results of such a pile loading test carried out at Swalwell are summarised in Figure 28 and are shown graphically in Figure 29. The load was applied to the head of the pile by jacking against a kentledge of large concrete blocks. This kentledge was supported on a platform which in turn was supported on four columns located well clear of the test pile. As the load was applied the vertical displacement of the pile head was measured by 4 No. spring-loaded deflection gauges fixed to a datum frame, this frame being supported on the ground and independent of both the test pile and the kentledge structure. The graph in Figure 29 shows that the load was applied in 4 No. incremental stages up to the working load and then unloaded in similar stages. Next the pile was loaded up to the proof load of 1½ x working load in 6 No. stages, held for 24 hours and then again unloaded in similar stages. At each stage of loading the load was held until the rate of displacement reduced to below 0.1mm in 20 minutes. A reading of the vertical recovery was also taken 24 hours after the unloading was completed.

A pile test is deemed to be satisfactory if the vertical displacement at proof load does not exceed 10% of the pile base diameter. At Swalwell the piles are 275 x 275mm square, hence a vertical displacement of 27.5mm would represent failure. In actual fact the maximum displacement at proof load was less than 4mm which indicates that the pile design is quite adequate.

5.3.3 GENERAL

As stated at the beginning of these notes on Swalwell D.P. the reason for including the two case studies was to illustrate the four stages of site investigation. Very few projects in fact contain all four stages but all projects should include the first and second stages, i.e. the preliminary and main ground investigation stages.

Remember that in the case of the pile test on a precast driven pile we are testing the ground's ability to support the pile rather than the integrity of the pile itself. Also in the case of monitoring the settlement of the storage facility, we are monitoring the behaviour of the ground due to the applied loading from the structure, which then affects the behaviour of the structure.

SWALWELL D.P. - GENERALISED STRATA SUCCESSION AND POSSIBLE FOUNDATION TYPES

TABLE 1

TABLE 1.

Depth to Top of STRATUM m	STRATUM NAME	Thickness m	Description of STATUM, and Properties	Possible Foundation Types
Surface Level	MADE GROUND	Av. 2 Min.0 Max.3	Shale, coal, rubble, ashes, topsoil, slag, clay, etc. - predominantly granular behaviour.	Shallow slab or raft, provided that:- (i) there is sufficient thickness of MADE GROUND at proposed construction site, (ii) uniform and fairly light loading can be achieved and/or some total & differential settlements can be tolerated
Av. 2 Min. 0 Max. 3	Organic silty CLAY.	Av. 7 Min.4 Max.10	Soft brown organic silty CLAY, with occasional thin pockets or bands of PEAT, - very weak and compressible.	Too weak and too compressible for any type of conventional foundation. Also could give rise to large settlements to slab/raft founded above. Ground improvement techniques(vibro-replacement, dynamic consolidation, surcharging etc.) might be technically feasible, but costly.
Av. 9 Min. 7 Max. 12	SAND and GRAVEL	Av. 3 Min.0 Max.8	Dense SAND AND GRAVEL, frequently clayey. N. values range 24-67, average 49. Thickness very variable, and occasionally stratum may be absent altogether.	May be suitable for bored or driven cast-in place concrete piles, especially if enlarged base formed (e.g.Frankipile). Average pile length say 10m. However, existence, thickness and density of stratum needs to be adequately proved at site.
Av. 12 Min. 9 Max. 16	Laminated CLAY	Av.10 Min. 2 Max.15	Firm(occasionally stiff) grey-brown laminated silty CLAY; with occasional gravel, cobbles and boulders, increasing in frequency with depth.	Not suitable for end-bearing or friction piles. Could cause excessive settlements to end bearing piles founded in SAND and GRAVEL above.
Av. 22 Min. 13 Max. 25	GLACIAL TILL	Av. 5 Min. 1 Max.13	Sandy silty CLAY (Boulder Clay), or clayey SAND and GRAVEL with COBBLES and BOULDERS. Varialbe thickness, N.values of 14 and 850.	Probably good stratum for founding piles, but at average length over 22m. will be very costly.
Av. 27+ Min. 20 Max. 34+	SANDSTONE Bedrock.	Not proved.	SANDSTONE - condition not proved.	Probably excellent stratum for founding piles, but again at average length over 27m. will be very costly.

FIGURE 20

SWALWELL D.P.

Existing Borehole Locations Obtained from Institute of Geological Sciences.

SCALE 1:2500

1 No. New Borehole Taken Here

Proposed New Corstruction

3 No. NEW BOREHOLES TAKEN HERE

PROPOSED NEW CONSTRUCTION

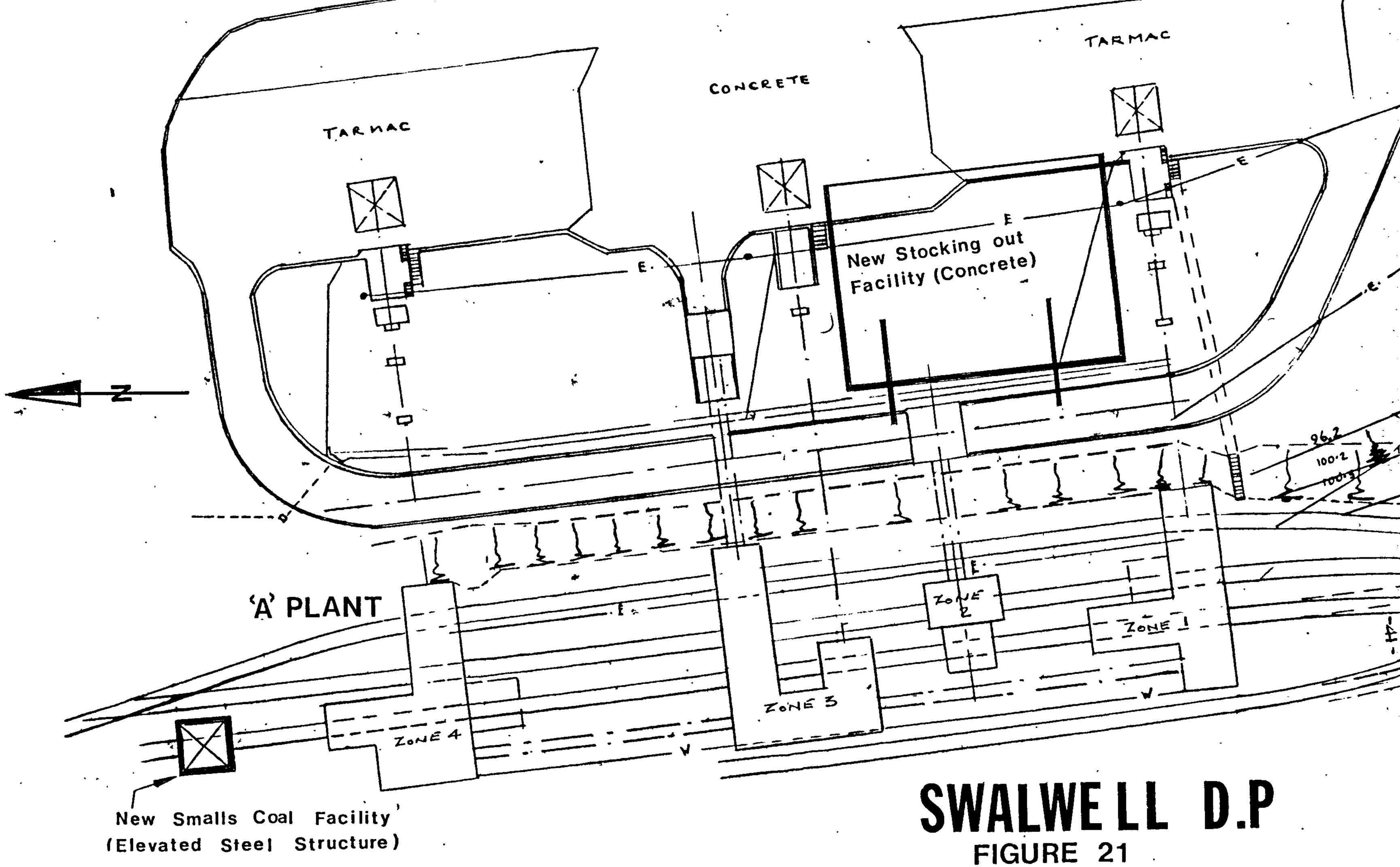
S W A L W E L L W A R D

Hannington Works (Engineering & Scrap Handling)

316 336 318 320 35 34 43A 303 43 305 304 42 298 299 300 303A 41

A B C D E F

N



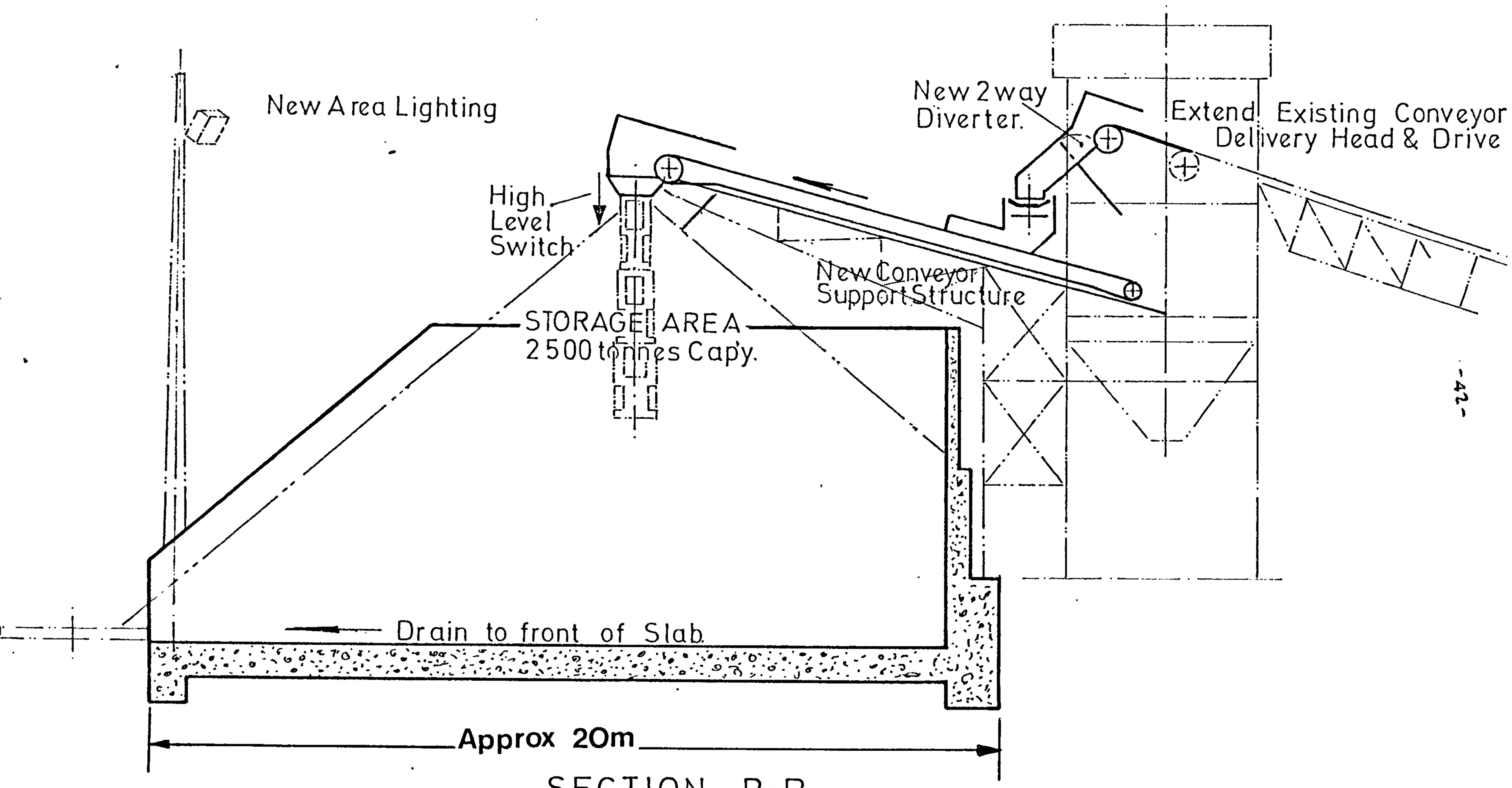
SWALWELL D.P

FIGURE 21

New Stocking out Facility
and
New Smalls Coal Facility

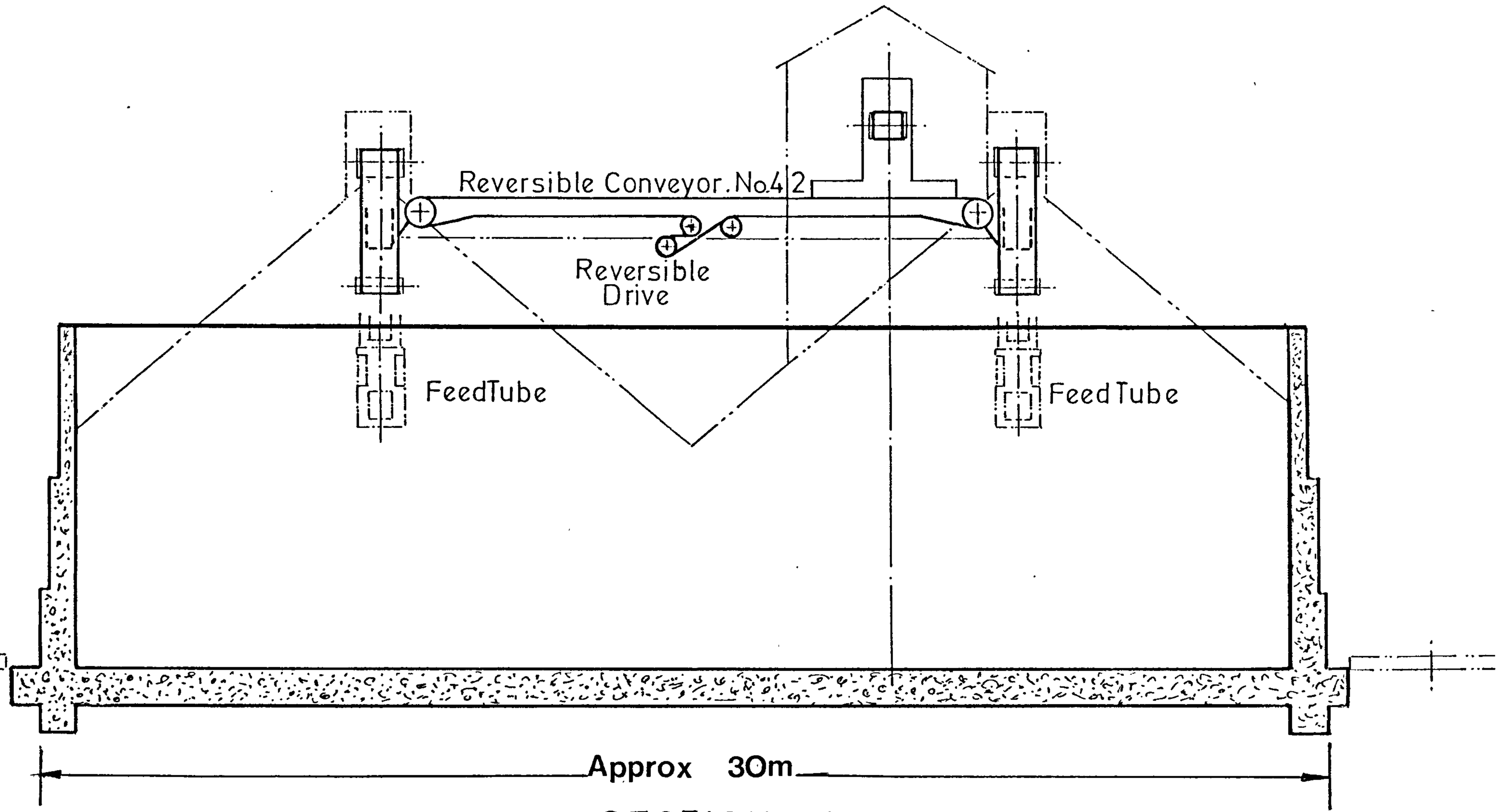
FIGURE 22

New Stocking Out Facility.



SECTION B-B

FIGURE 23
New Stocking Out Facility.



SECTION A-A

SWALWELL D.P. — NEW STOCKING OUT FACILITY.

FOUNDATION CALCULATIONS FOR SLAB

FIGURE 24

BEARING CAPACITY

SETTLEMENT

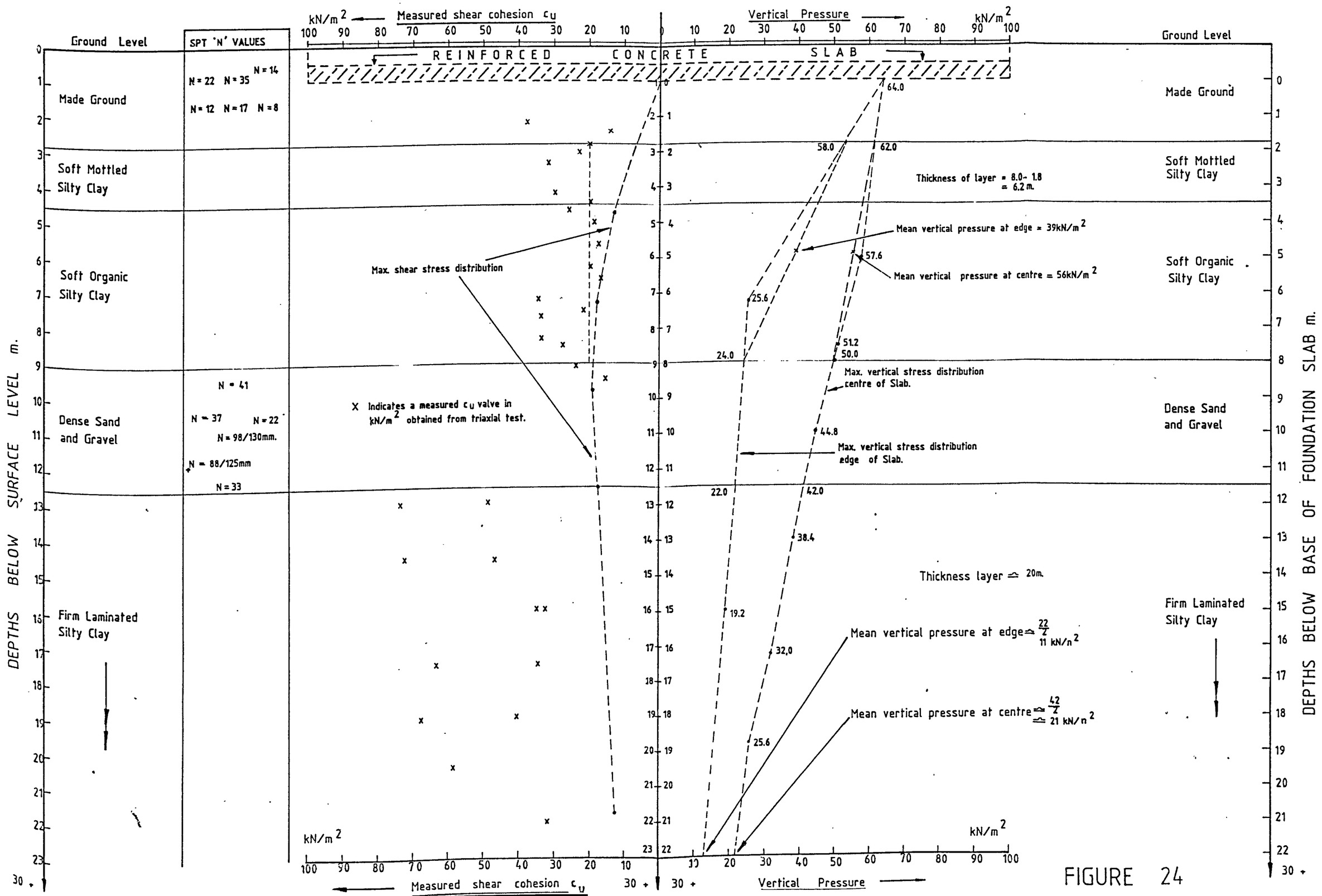
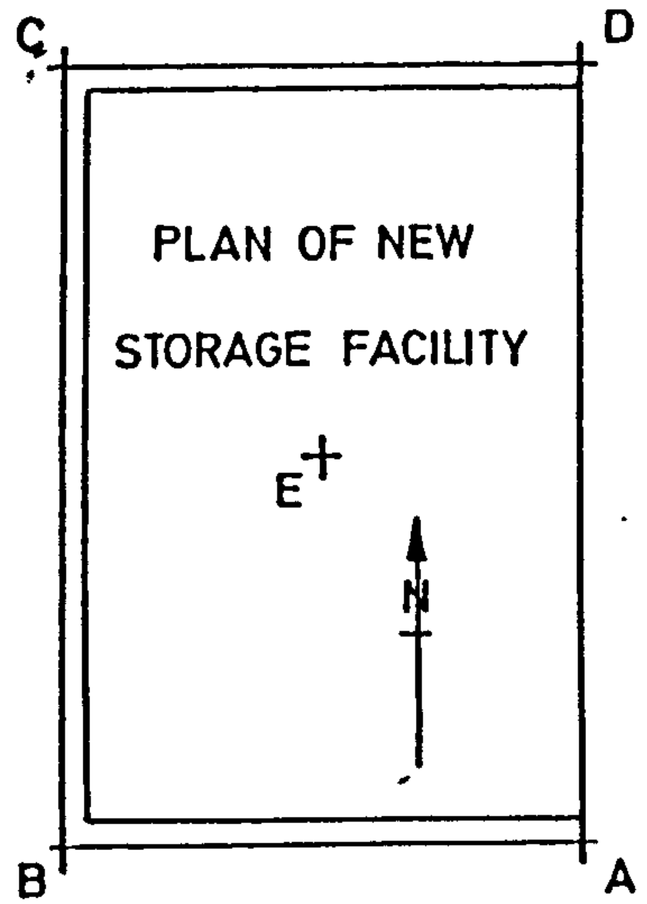


FIGURE 24

SWALWELL D.P. SETTLEMENT OF BASE OF STORAGE FACILITY

FIGURE 25



SETTLEMENT IN mm

Levelling of Base Reference Points Commenced 19/7/82

Predicted Settlement at Western Edge and Centre of Base (B, C, E)

Predicted Settlement at NE and SE Corners of Base (A, D)

C
E
B
D
A

300
250
200
150
100
50
0

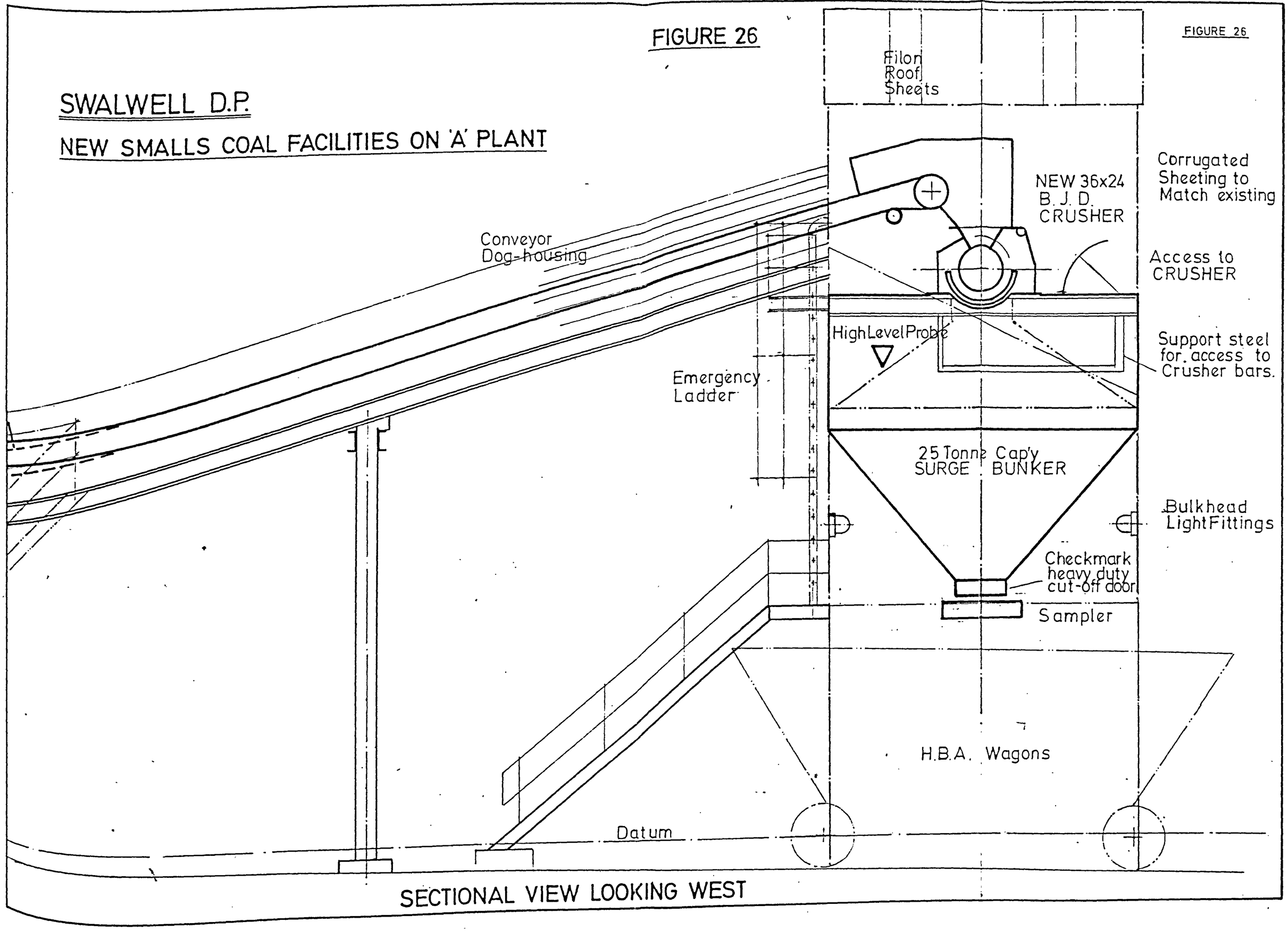
4/82 6/82 12/82 6/83 12/83 6/84 12/84 6/85 12/85 6/86 12/86 6/87 12/87 6/88 12/88 6/89 12/89 6/90

TIME ELAPSED IN MONTHS

FIGURE 26

SWALWELL D.P.

NEW SMALLS COAL FACILITIES ON 'A' PLANT



SECTIONAL VIEW LOOKING WEST

N (SPT) value in blows/300mm.
Undrained Shear Strength cu in kN/m²

Stanchions

Pile Cap | Pile Cap

MADE GROUND	N = 37
	N = 56
	N = 20
Soft Sandy CLAY	Cu = 15
Soft Organic Silty CLAY Becoming very peaty	Cu = 40
	Cu = 33
	Cu = 23
	Cu = 13
Dense SAND and GRAVEL	N = 58
	N = 46
Firm Laminated CLAY	Cu = 23
	Cu = 67
	Cu = 44
	Cu = 38
	Cu = 60
	Cu = 71
	Cu = 73
	Cu = 57
Stiff Laminated CLAY	Cu = 97
	Cu = 110
	Cu = 70
	Cu = 64
	Cu = 97
Bands of Stiff Sandy CLAY and very dense SAND. GRAVEL and COBBLES	N = 26
	N = 56
	N = 107
Weathered SANDSTONE	N* = 50/42mm.

Max. Standing Water Level
SO_3 = 0·40 pp 10^5
pH = 7·8

10 20 30 40 kN/m² Vertical Pressure

Vertical Pressure Distribution.

Final Standing Water Level
(with casing @ 37·30m.)

DEPTH BELOW SURFACE IN METRES

SWALWELL D.P.

NEW SMALLS COAL FACILITY ON 'A' PLANT - PILED FOUNDATIONS.

FIGURE 27

Piling Engineers and Contractors

FIGURE 28

BALKEN PILING LIMITED

MAIN ROAD, PYE BRIDGE, DERBY DE5 4PY. TELEPHONE: LEABROOKS 3091-2-3 TELEX: 377424

Dear Sirs,

We enclose the following pile test documents:-

(1) SCHEDULE OF DEFLECTION GAUGE READINGS
(2) LOAD DEFLECTION CURVE
(3) LOAD AND DEFLECTION ON A TIME BASE

SUMMARY OF TEST RESULTS

CONTRACT SWALLWELL

CONTRACT NO. 320489

PILE NO. ONE

TEST NO. ONE

WORKING LOAD 221 KN

TEST LOAD 331·5 KN

PILE LENGTH 10·800m

FINAL SET 29mm/20Blows.

HAMMER WEIGHT 30 KN

HAMMER DROP 300mm

DEFLECTION AT WORKING LOAD 2·60mm

RESIDUAL DEFELCTION FROM WORKING LOAD 2·19 mm

DEFLECTION AT TEST LOAD 3·92 mm

RESIDUAL DEFLECTION FROM TEST LOAD 3·70mm

% RECOVERY 94%

MAXIMUM ELASTIC DEFLECTION
AT WORKING LOAD 0·9mm
AT TEST LOAD 1·35mm.

DATE TEST STARTED 6/8/82

DATE TEST COMPLETE 9/8/82

TEST ENGINEER N Berry

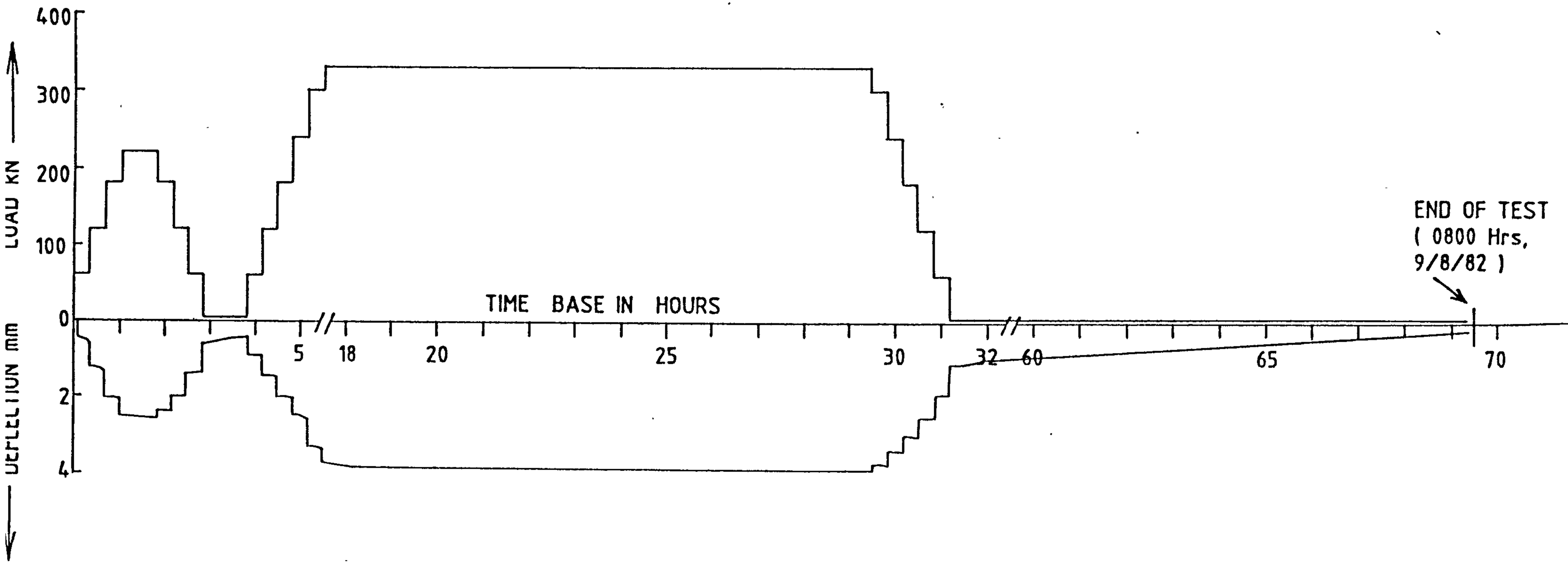
A MEMBER OF THE BALKEN GROUP OF SWEDEN

Directors: B. GRAVARE (SWEDEN) CHAIRMAN, M. J. SANDS MANAGING, A. C. BADHOLM (SWEDEN)
Secretary: R. KIRK, A.C.M.A.
Registered in England at Main Road, Pye Bridge, Derby. Registration No 845182

FIGURE 29

SWALLWELL DP

NEW SMALLS COAL FACILITY ON 'A' PLANT
LOAD / DEFLECTION RELATIONSHIP FOR
TEST ON 275 x 275mm square DRIVEN
PRECAST CONCRETE PILE.



6. UNDERGROUND WORKINGS

6.1 Site Investigations for Old Underground Workings

Old underground workings (both pillar and stall, and longwall types) are a frequent occurrence at opencast coal sites in Britain. Proving of these old workings in a potential excavation area is part of the normal prospecting operation using openhole air-flush rotary drilling equipment.

Where the presence of shallow old workings is to be investigated for a proposed structure or other development the same method of drilling as used for coal prospecting is suitable. Also openhole drilling using rotary-percussive techniques is equally suitable.

The use of down-the-borehole video cameras is very useful for examining and recording the condition of old workings, and can be of assistance in choosing the best method of filling the void spaces if this is deemed necessary.

Where shafts or bell-pits etc. are being investigated cable percussion boring is often the best method, particularly if the shaft has been filled or partially filled. Also where the rockhead is very shallow, say up to 5m deep, then a backhoe or small dragline is sometimes used. Whenever site investigations to find old shafts are being carried out, it is essential that measures are taken to ensure the safety of men and equipment, in case of rapid collapse of the filling or capping, (e.g. provision of platforms, anchorages, harnesses etc).

6.2 Edmondsley O.C.C.S. - Grouting Old Workings Under H.P. Gas Mains

Edmondsley is a relatively small County Durham opencast site which has recently been restored. In the vicinity of the site are two high pressure gas transmission pipelines, these being a 760mm dia. 69 bar line running north-south across the site area, and a 320mm dia. 19 bar line running east-west along the northern boundary. These high pressure gas lines are potentially very dangerous should a fracture or rupture of the pipe occur, and because of public concern about this the Opencast Executive entered into early and extensive discussions with Northern Gas about protecting these installations. The discussions led to a firm of Consulting Engineers being appointed by Northern Gas to design and supervise the necessary works to ensure the safety of the two gas lines.

The protection measures proposed by the Consulting Engineers are shown schematically in Figure 30, and summarised as follows:-

(i) 30m minimum stand-off from ℄ of gas pipeline to Limit of Excavation,

(ii) 1 in 1 (45°) overall batters,

(iii) fenced exclusion zone for 10m either side of gas pipeline ℄ ,

(iv) vehicle crossings only via special concrete pads,

(v) old workings to be grout filled wherever depth from rockhead to roof of workings is less than 10x assessed roadway width,

(vi) no blasting.

It was required to stabilise the old workings beneath three sections of pipeline, two sections being under the 760mm dia. pipe and one section under the 320mm dia. pipe. The occurrence of the shallow old workings had been investigated during the normal prospecting operations using the usual open-hole techniques. No special or further investigations were carried out in the areas where grouting was proposed.

The scheme for filling the old workings voids involved openhole drilling to 50mm dia. and grouting with a cement/P.F.A./water mixture. The spacings and sequence for vertical and inclined boreholes are as shown on Figure 31. The intention was to grout-fill only a narrow strip of ground (minimum width 6m) vertically below the pipeline.

Once this project had got underway it was found that the grout-take was much greater than had been anticipated. In the end the quantity of grout used was about 3½ times the amount allowed for. The contract period had to be extended from 5 weeks to 13 weeks, and the final cost of the project was about 3 times the estimate.

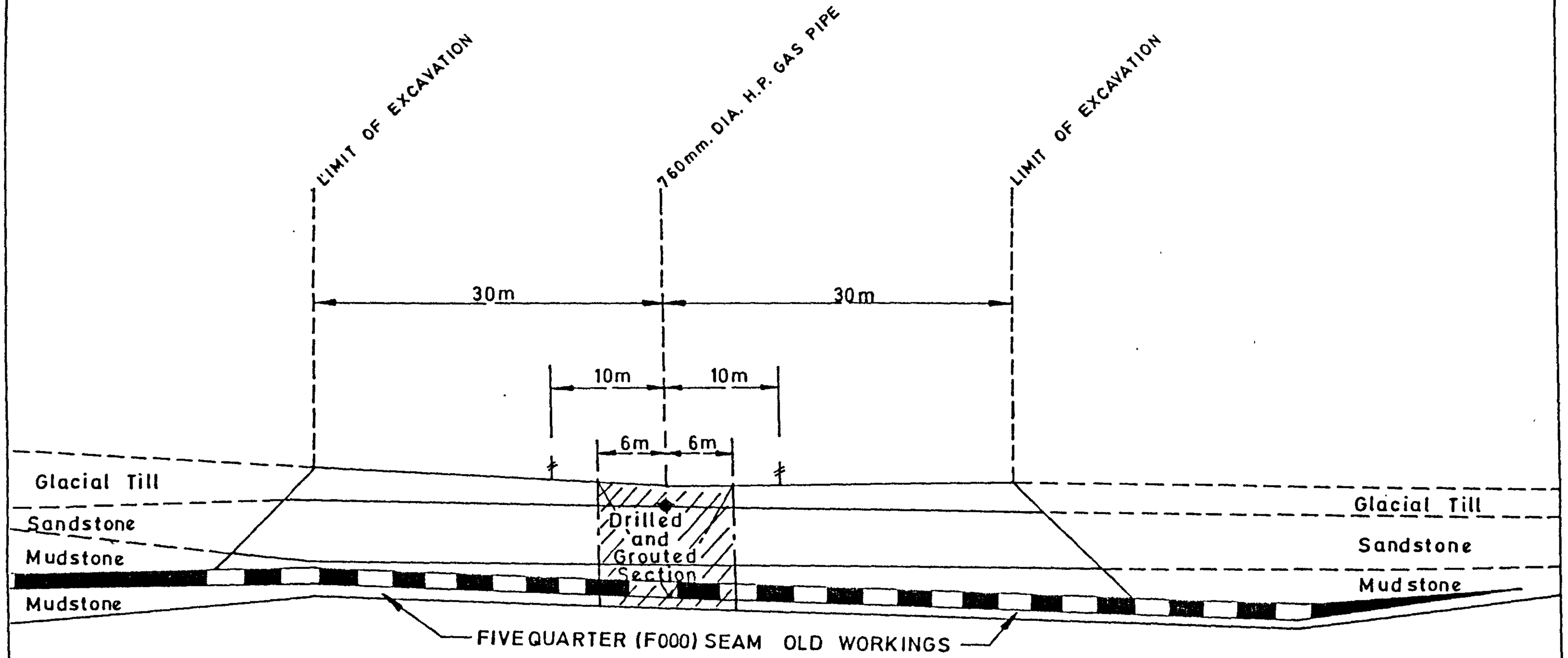
The contract correspondence states that the increase in grout quantity was "due to the presence of more extensive old mine workings than anticipated from site drilling information available at the time of tender." This seems to assert that the site investigation was not adequate. In view of the fact that no specific investigations of the old workings in the areas to be grouted were carried out, this assertion seems reasonable. However, it should be pointed out that no steps were taken to prevent the lateral spread of the grout beyond the section of old workings which it was desired to stabilise, and so this could also account for some of the large additional quantities of grout used.

When grouting old workings it is normal practice to construct a barrier or cut-off at the boundaries of the area which it is required to treat, and so prevent the lateral spreading of the grout. This barrier may be formed by drilling closely spaced boreholes around the perimeter to be treated, and then injecting a dry granular material such as pea-gravel. This procedure causes the formation of cones of granular material in the void spaces under each borehole, and so forms a continuous granular barrier along the line of perimeter boreholes. The flow of grout is arrested on meeting this barrier, and is thus confined to that area which it is required to fill.

6.3 Recent Developments in Filling Old Workings - Rock Paste

A new method for completely filling abandoned mine workings has recently been developed by Ove Arup and Partners (Ground Engineering, May 1983). The method is to fill the old workings with waste rock materials mixed with water to the consistency of a thick non-setting paste, called rock paste, and which can be pumped into the ground. The theory is that the rock paste, if mixed to the correct consistency, will have considerable spreading properties and will spread under its own weight and due to the pumping impulses. Placing will be via conventional concrete pumping equipment from a surface batching site, and fed into holes drilled vertically down into the workings. Pipelines of up to 250mm dia. discharging at rates of about 100m^3 per hour are envisaged and pumping trials are currently in progress at the Building Research Establishment at Garston. It is anticipated that there will be considerable cost savings against conventional grouting methods, due to the exclusion of cement and the use of low cost waste materials.

FIGURE 30 EDMONDSLEY O.C.C.S. 1/500



TYPICAL SECTION UNDER 760mm. DIA. HIGH PRESSURE (69 bars) GAS TRANSMISSION LINE

FIGURE 30

FIGURE 31

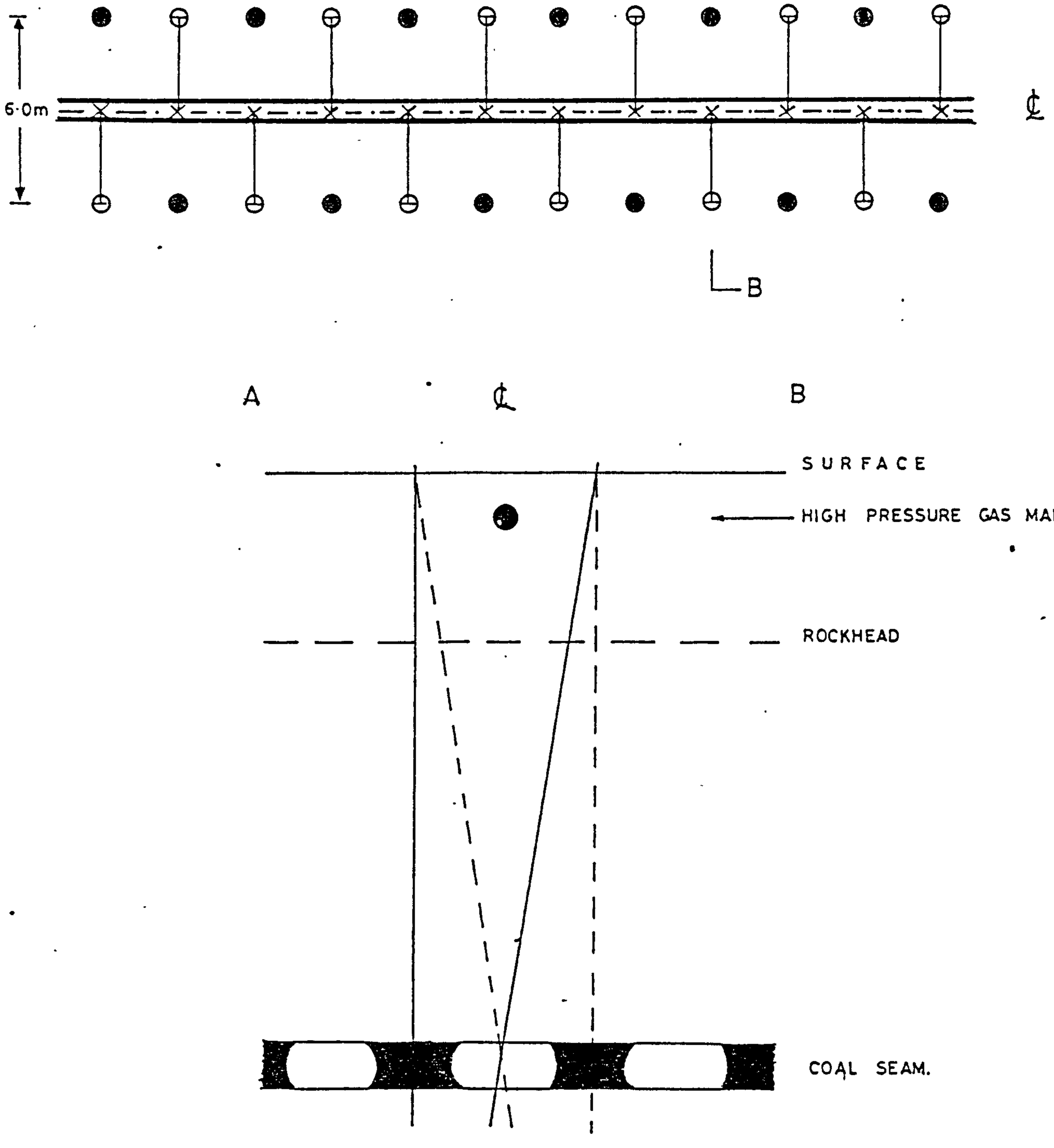


FIGURE 31

NATIONAL COAL BOARD, OPENCAST EXECUTIVE

EDMONDSLEY SITE, CO. DURHAM.

DRG. No. 3/4016/1

SCALE:- 1:200 NAT.

GAS TRANSMISSION LINES
STABILISATION OF OLD MINE WORKINGS.
LAYOUT OF GROUT INJECTION BOREHOLES.

REFERENCE.

	VERTICAL	INCLINED
PRIMARY BOREHOLES. . . . . SHOWN	●	⟶×
SECONDARY BOREHOLES. . . . . SHOWN	○	

MARCH 1980

7. CONCLUSION

Site investigation is a very large subject, and these notes should only be seen as a very brief introduction. However, it is hoped that engineers who are planning new works or structures, or extensions and alterations at existing sites, will recognise the part that a site investigation should play in the planning and design of all construction work which involves digging through or imposing loads on the ground.

Further reading in site investigations, and geotechnical engineering generally, can be found in the British Standard Codes given in Section 1.4 of these notes, and in the extensive reference lists given in those Codes.

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Regional Geotechnical Engineer, North East Region.

July 1983.

PAPER C6 DERWENT–TYNE CONFLUENCE

Hughes, D.B. 1986.

Ground investigations and foundations in deep alluvial deposits near the Derwent–Tyne confluence.

Journal of the Institute of Science Technology, No.16, March 1986, 14-21.

GROUND INVESTIGATIONS AND FOUNDATIONS IN DEEP ALLUVIAL DEPOSITS NEAR THE DERWENT-TYNE CONFLUENCE

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National Coal Board, Opencast Executive

INTRODUCTION

The confluence of the River Derwent and the River Tyne occurs in the County of Tyne and Wear and is located to the west of Newcastle upon Tyne. Figure 1 shows the National Coal Board's Swalwell Disposal Point (D.P.) as being immediately to the south-east of the confluence of the two rivers, and shows the locations of some earlier major construction projects in the near vicinity; including Gateshead Western By-pass, Scotswood Bridge, and Dunston Power Station, all of which have been in existence for several years. Also shown is the Gateshead Metrocentre which is currently under construction, and the proposed Newcastle Western By-pass for which ground investigations have been carried out recently.

SWALWELL COAL DISPOSAL POINT

About one million tonnes per annum of opencast mined coal is brought into Swalwell D.P. by road from sites located south of the River Tyne. Here it is blended, screened and crushed, and then transported to customers by either road or rail.

The disposal point at Swalwell has been in existence since 1946, and has undergone numerous extensions and modifications over the years. In 1982 two new structures were erected at the site, and this paper is mainly concerned with the ground investigations and foundation aspects of these new coal handling facilities.

Storage Bunker

This 2,500 tonne capacity storage bunker was required to store coal which had been processed through the screening and crushing plant for later loading out onto rail transport. The completed reinforced concrete structure is shown in Figure 2. It consists of a base slab at ground level which is approximately 30m x 20m, and has retaining walls of up to 8m height on three sides. A high level conveyor system discharges coal into the storage void from a height of about 11m above the base slab.

Smalls Coal Plant

The purpose of this plant is to process "run of mine" coal by screening and crushing, and then to outload "smalls" coal into trains of rail wagons at a rate of 150 tonne per hour. The plant consists of a rising conveyor which feeds into an elevated crusher. The crushed coal then falls into a 25 tonne capacity surge bunker, which in turn discharges into rail wagons as they pass beneath. The whole of the superstructure is fabricated in steelwork, is supported on four steel stanchions, and stands to a maximum height of 13m above the track level. The completed structure is shown in Figure 3.

GROUND INVESTIGATIONS

Existing Records

No records for any previous foundation works or ground investigations were available for any of the existing structures at the D.P.

The British Geological Survey 1:10560 sheet (1) for the area shows the site to be underlain by about 33m thickness of alluvium lying on Lower Coal Measures bedrock. Further information supplied by the B.G.S. included borehole records obtained for some of the previous construction projects in the area. These records indicated that there was a probability of soft compressible organic silty clay and laminated clay beneath the sites for the proposed structures.

Hence difficult foundation conditions were anticipated, and a direct ground investigation exercise was planned with the object of proving the ground conditions.

Main Ground Investigation

In October 1981 three boreholes were sunk by cable percussion methods at the site for the storage bunker. The first borehole went to over 31m but did not fully penetrate the alluvial deposits, whilst the second and third boreholes were terminated at less depth. Later, in March 1982, a fourth cable percussion borehole was sunk at the site for the smalls coal plant, and this did prove the full depth of the alluvium finding bedrock at about 37m. All boreholes were cased for their full depth.

Open-drive 100mm diameter tube samples were obtained from throughout the cohesive strata; also in situ standard penetration tests were carried out in granular strata and weathered bedrock. In the laboratory, soil mechanics tests to B.S. 1377(2) were carried out on the samples and included moisture content determination, particle size distribution (grading), undrained shear strength (triaxial), and consolidation (oedometer) tests.

The results of this investigation are summarised in Figure 4. It can be seen that a layer of about 3m thickness of made ground covers the whole area and is underlain by some 7m of soft organic silty clay, followed by a 3m thick layer of dense sand and gravel. Beneath the sand and gravel is about 19m of firm laminated silty clay containing occasional pockets of sand, followed by 5m or so of glacial drift, with sandstone bedrock at about 37m below surface level.

The ground conditions as proved by the cable percussion boring exercise generally confirmed the conditions indicated by the geological plan and previous borehole records. The organic silty clay and the laminated silty clay, both being highly compressible and having low shear strength, represented serious foundation design problems. However, the 3m thick layer of made ground at the surface, and the dense sand and gravel at around 10m to 13m depth, also offered possibilities for overcoming these problems.

FOUNDATION DESIGNS AND MONITORING

Foundations for Storage Bunker

Apart from the overhead conveyor connection, the storage bunker is completely independent of all other structures and services. Hence large settlement of the base can readily be accommodated. The bunker is founded near the top of the 3m thick made ground layer, and the calculated Factor of Safety (FoS) against bearing failure is 2.7, i.e. just adequate (CP2004(3)).

Calculations based on oedometer test results and Boussinesq's theory (4) indicated that, due to the eccentric loading of the bunker, the west side (points B and C) would settle about 275mm ultimately, whereas the east side (points A and D) would only go down a total of 90mm. This settlement being due to consolidation of both the organic silty clay and the laminated clay. It was also calculated that 90% of the total settlements would be complete within about 5 years of initial loading.

Settlement Monitoring

Survey stations on each of the four outer corners of the base slab have been monitored regularly since the date of the first full loading of the bunker, and the results of this exercise are shown in Figure 5. It can be seen that whilst the predicted magnitude of settlements for points B, C and D are reasonably close, point A has gone down rather more than expected. Also, the initial rate of settlement was far ahead of the predicted rate, but has subsequently slowed down very considerably.

Foundations for Smalls Coal Plant

A piled foundation was chosen for the smalls coal plant, as this elevated structure was required to span a railway siding, and therefore large settlements of the foundations could not be tolerated. The 3m. thick layer of dense sand and gravel lying some 10m. below the surface was considered to be a suitable founding stratum for end-bearing piles. Eight piles of pre-cast concrete construction were driven (2 under each stanchion) and all attained the designed set within the dense sand and gravel layer.

Calculations for the pile group indicated that the eventual total settlement will be 27mm and this should be 90% complete within 25 years of initial loading of the structure. The whole of this settlement should result from consolidation of the underlying laminated silty clay.

Pile Test

In order to confirm the suitability of the piles a proof load test was carried out on one of the working piles, and the sequence of loading and corresponding pile head deflections are shown in Figure 6.

The load was applied to the head of the pile by jacking against a kentledge of large concrete blocks. This kentledge was supported on a platform which in turn was supported on columns located well clear of the test pile. As the load was applied the vertical displacement of the pile head was measured by spring-loaded deflection gauges fixed to an independent datum frame.

Figure 6 shows that the load was applied in 4 incremental stages up to the working load and then unloaded in similar stages. Next the pile was loaded up to the proof load of 1½ x working load in 6 stages, held for 24 hours and then again unloaded in similar stages. At each stage of loading the load was held until the rate of displacement reduced to below 0.1mm in 20 minutes. A reading of the vertical recovery was also taken 24 hours after the unloading was completed.

A pile test is deemed to be satisfactory if the vertical displacement at proof load does not exceed 10% of the pile base diameter (CP2004(3)). At Swalwell the piles are 275 x 275mm square, hence a vertical displacement of around 25 - 30mm. would represent failure. In fact the maximum displacement at proof load was less than 4mm which indicated that the pile design was quite adequate.

CONCLUSIONS

The details from a conventional ground investigation exercise have been presented for a site underlain by deep alluvial deposits. Different foundation types were adopted for the two structures, and their performance has been monitored.

For the slab foundation of the storage bunker total settlement predictions are proving to be fairly accurate at 3 of the 4 monitoring stations, but the rate of settlement was somewhat underestimated. A possible explanation for this is that soil fabric and drainage characteristics of relatively soft alluvial clays are disturbed or not represented in fairly small-size normal sampling and testing procedures, hence the rate of consolidation tends to be greater than that measured in the standard oedometer test. Larger sized (up to 250mm diameter) piston sampling and consolidation testing are described by Rowe (5).

For the smalls coal plant precast concrete piles were driven into a layer of dense sand and gravel some 10 to 13m below the surface. On completion of pile-driving a loading test was carried out on one of the working piles and this gave very satisfactory results.

ACKNOWLEDGEMENTS

The Author wishes to thank the N.C.B. Opencast Executive for permission to present this paper. Also to thank the Executive's surveying staff for their part in the monitoring work.

The following firms have been involved in the projects:-

Norwest Holst Soil Engineering Ltd. - ground investigations
Don Valley Engineering Ltd. - design and construction of storage bunker
B. & K. Steelwork Fabrications Ltd. - design and construction of smalls coal plant
Balken Piling Ltd. - installation of piles, and pile test.

The views expressed in this paper are entirely those of the Author.

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(1) Geological Survey of Great Britain (England and Wales); Scale 1:10560; Sheet NZ 26 SW (Durham - Northumberland); Ordnance Survey, 1976.

(2) British Standards Institution; 'Methods of Test for Soil for Civil Engineering Purposes'; BS 1377: 1975.

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(4) Boussinesq, J.; 'Application des potentials à l'étude de l'équilibre et de mouvement des solids élastiques'; Paris, 1885, (Gauthier-Villars).

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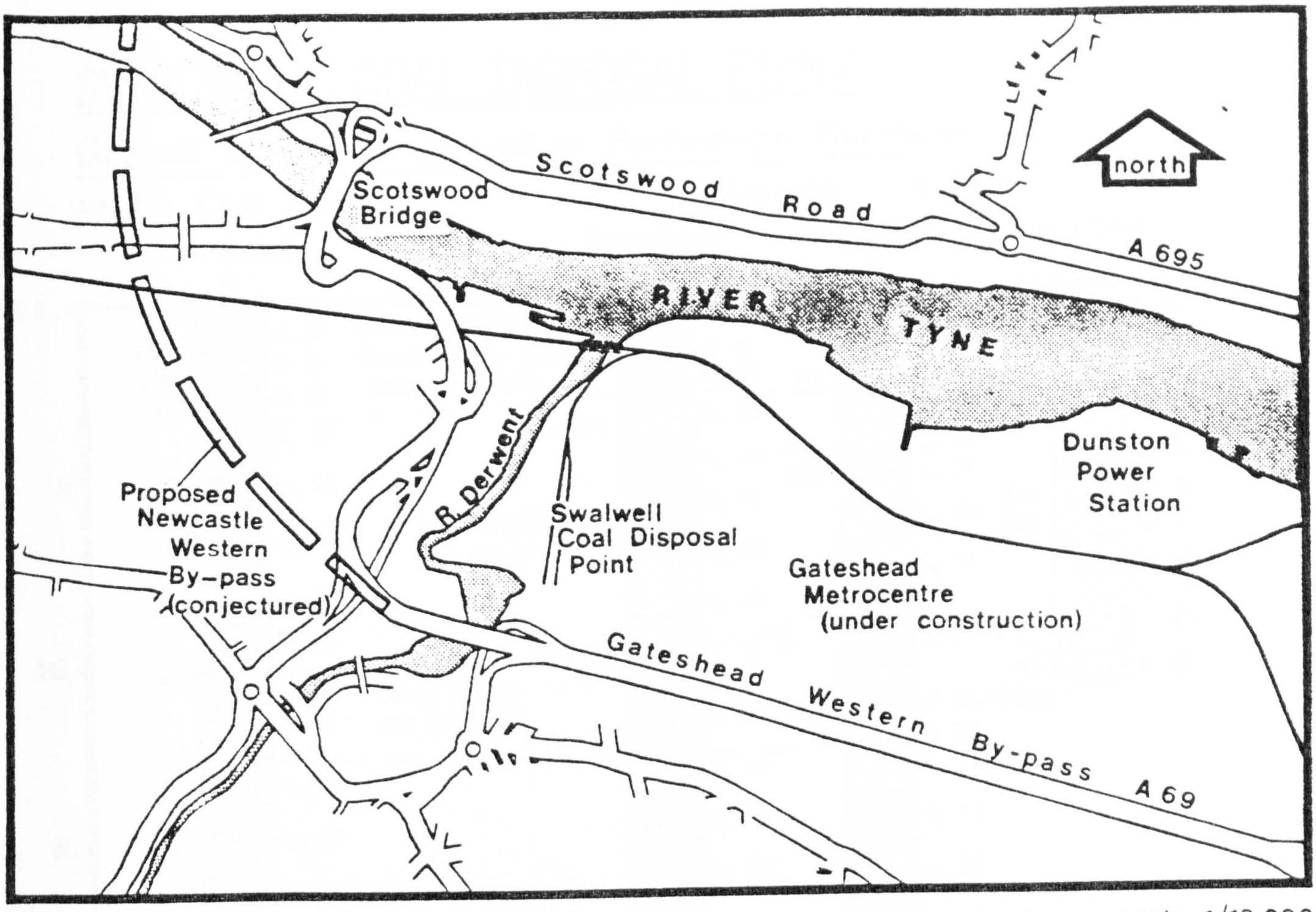
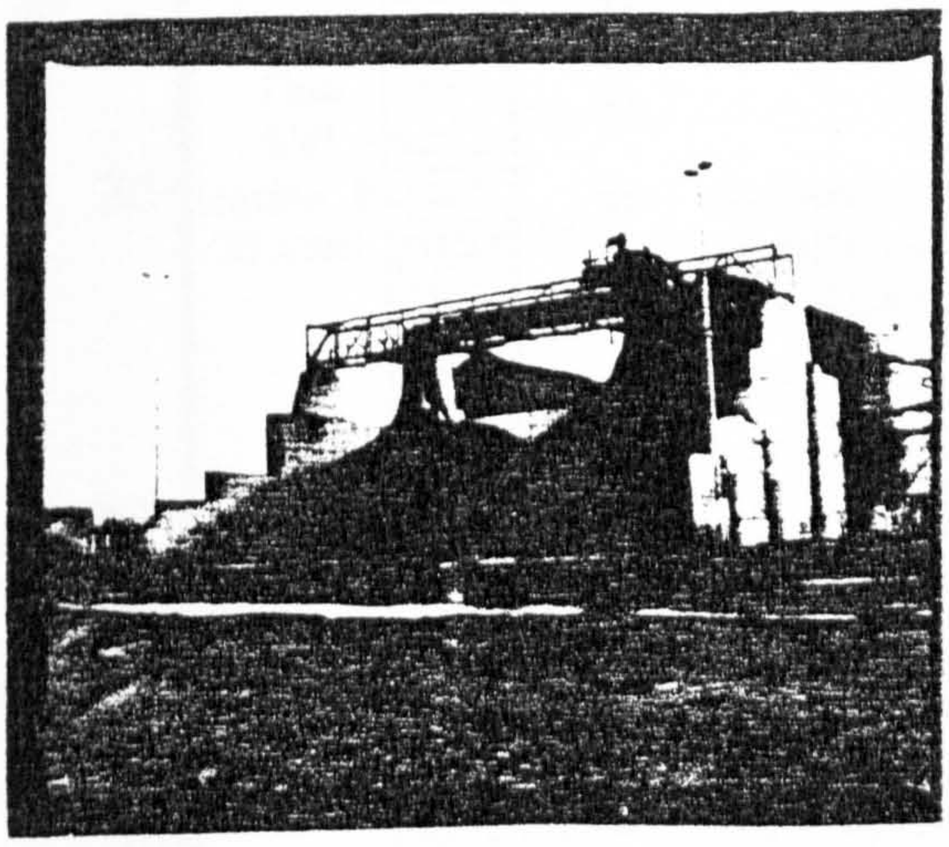
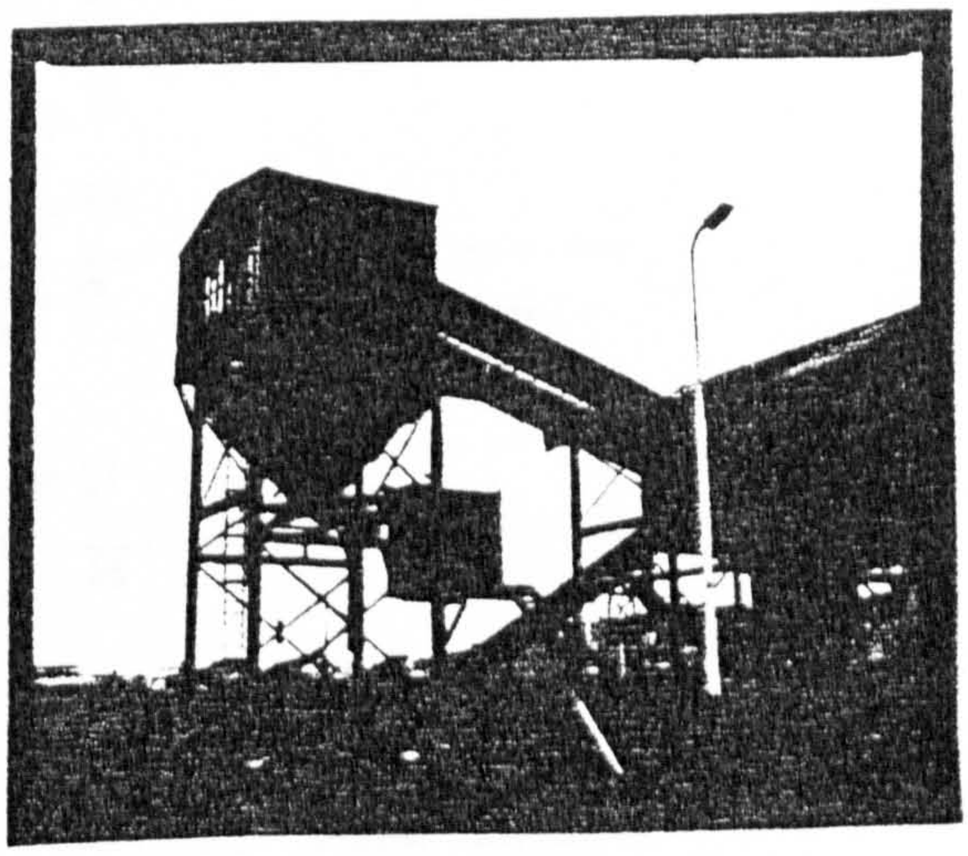


Figure 1. LOCATION PLAN Approx. scale 1/18 000



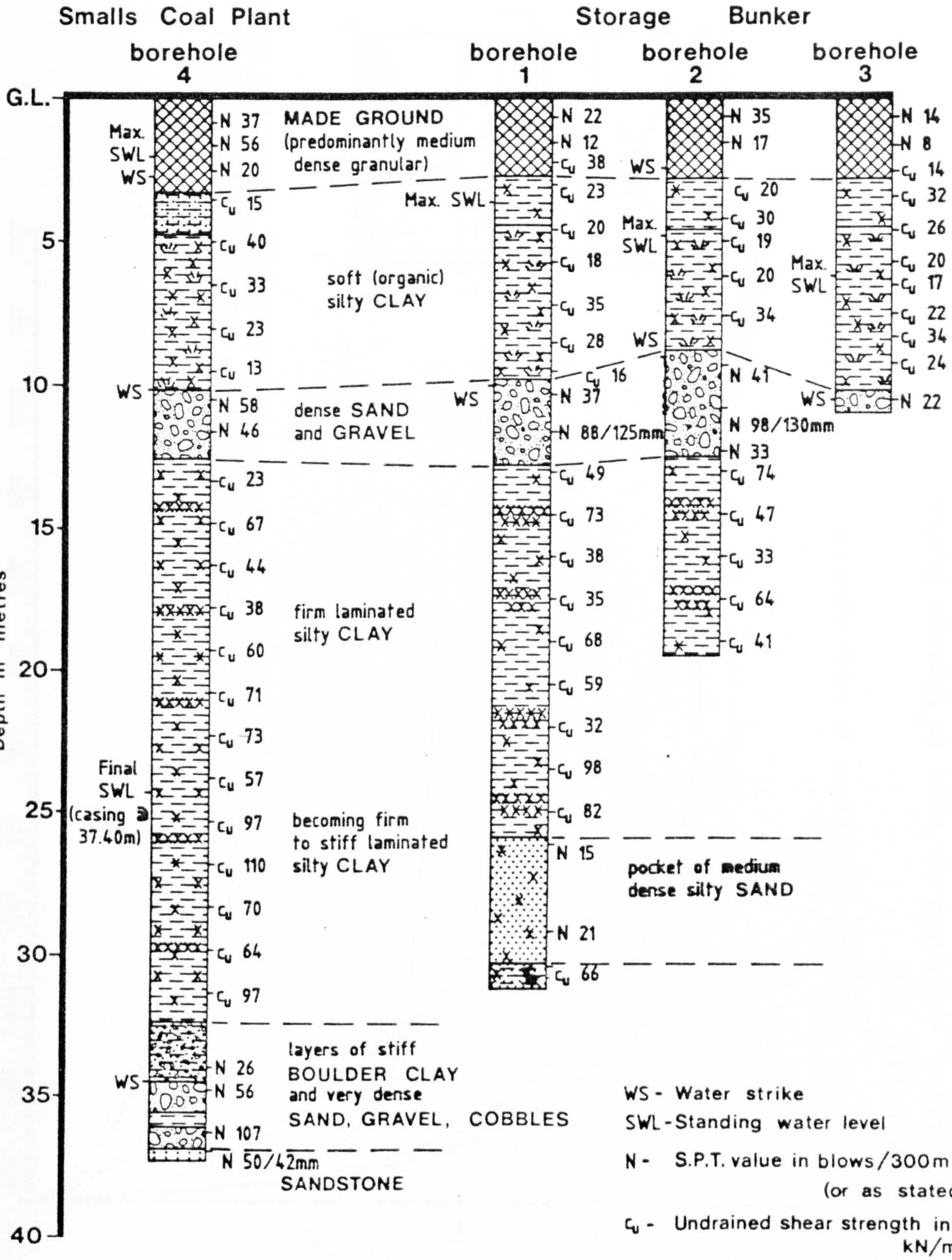
STORAGE BUNKER
Figure 2.



SMALLS COAL PLANT
Figure 3.

SWALWELL COAL DISPOSAL POINT

Ground conditions proved in Percussion Boreholes.



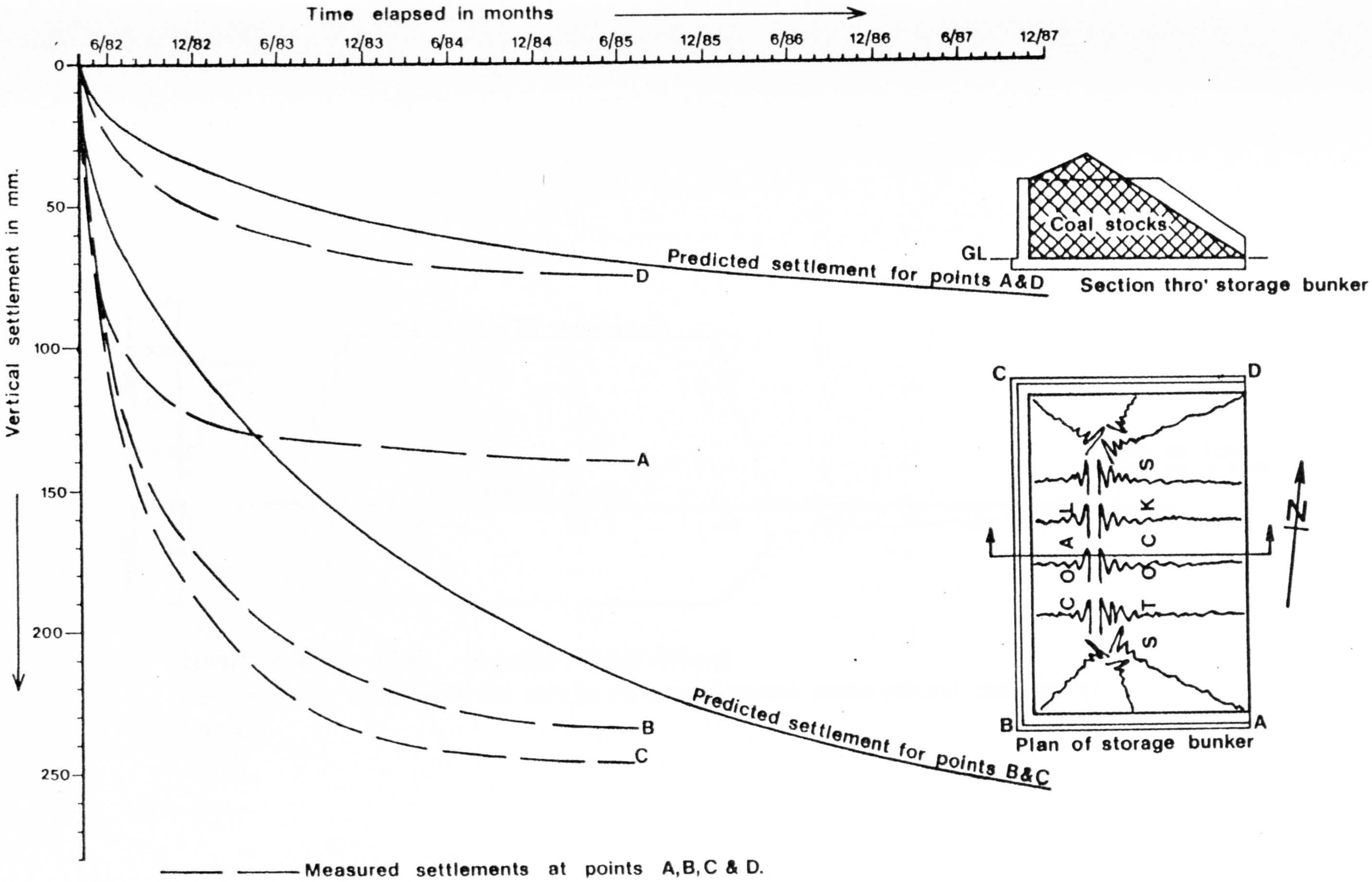
WS - Water strike

SWL - Standing water level

N - S.P.T. value in blows/300mm (or as stated)

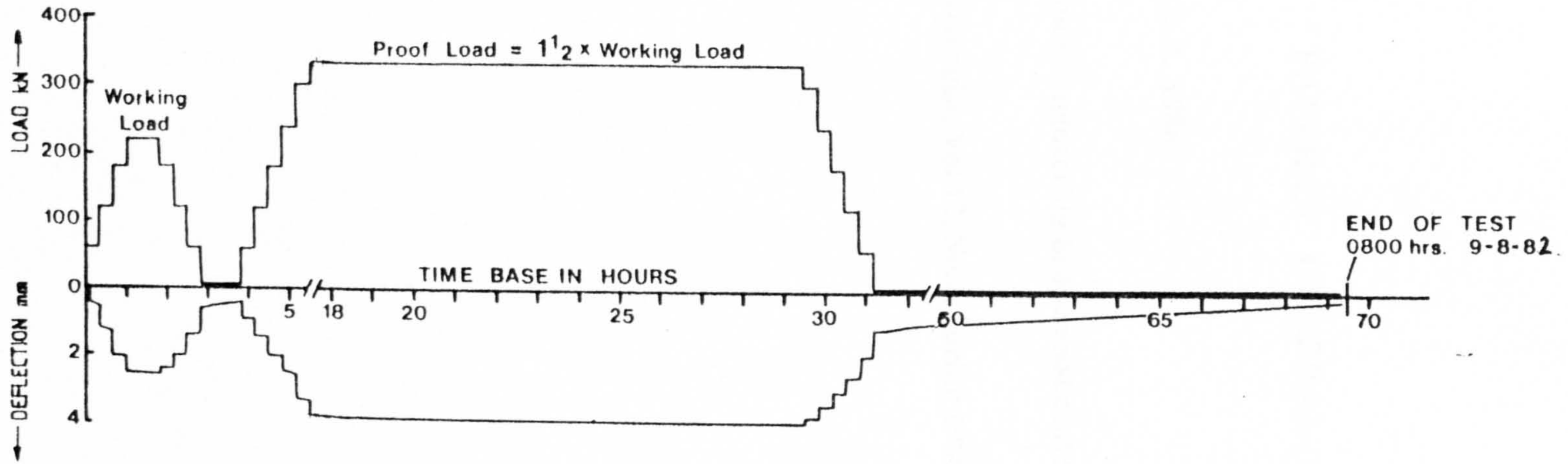
c_u - Undrained shear strength in kN/m

Figure 4.



SWALWELL D.P.
Settlement measured on base of Storage Bunker.

Figure 5.



SWALWELL D.P. Smalls Coal Plant

LOAD/DEFLECTION RELATIONSHIP FOR TEST ON 275 x 275mm SQUARE DRIVEN PRECAST CONCRETE PILE

Figure 6

PAPER C7 GENERAL (B.G.)

Hughes, D.B. 1986.

Geotechnical engineering in opencast coal mining.

British Geologist, Vol.12, No.2, Spring 1986, 61-63.

data sets—perhaps including site investigation reports held by, say, local authorities as discussed by Raybould and Kenna in this issue of *British Geologist*.

Information Services

An enquiry service is operated in the Keyworth and Edinburgh offices of BGS to help the enquirer obtain the appropriate scientific advice or service and to act as a formal route to the facilities offered by the NGDC. An Information Office has also been established in the Geological Museum, London, to offer a local service there and staff in other BGS offices have been appointed to act in a similar capacity.

In addition to information about the data collections held by BGS, members of the public may also obtain details of formal publications issued by the Survey (maps, memoirs, books and reports) and order copies through a mail order service. This service also deals with the processing of orders for copies of unpublished material such as open-file reports, paper dyeline maps and data packages, and arrangements are being made to issue bi-monthly lists of all documentary material produced by the Survey.

Non-confidential material held in the NGDC archives may be consulted directly by members of the public provided prior appointments are made for this purpose. Microfilm copies of documentary material will be presented for inspection although electrostatic or photographic copies of original pages, sheets or reports may be prepared on demand. Core in the borehole archive may also be examined and, provided certain conditions are satisfied, may be sampled for analytical or testing purposes. One of the major conditions is that the results of analyses or determinations are presented to the NGDC for inclusion in the relevant database or collection and, after a limited period of confidentiality if necessary, can be made available to other enquirers. Thus the national archive benefits from such "loans". Charges may be made for those services requiring the use of staff resources, and copying costs are levied at standard rates; details are given in a separate information leaflet.

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BRIAN KELK

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The Applied Geophysics Unit at University College Galway offers a one year masters degree in applied geophysics. The course involves lectures, practicals, and a field project, and is designed to emphasise exploration principles. Applications are invited from suitably qualified persons with good first degrees in geology or physics. The closing date for the course which begins in October 1986 is 30th June 1986. Further information and application forms may be obtained from Prof. A. Brock, Applied Geophysics Unit, University College Galway, Ireland.

The Scope for RENEWABLE ENERGY in the UK

This 'state of the art' report by the British Renewable Energy Forum covers all the main RE sectors including geothermal energy. Copies of this 46-page booklet can be obtained from the Institution of Geologists office (post free) at a cost of £3, payment with order please.

GEOTECHNICAL ENGINEERING IN OPENCAST COAL MINING

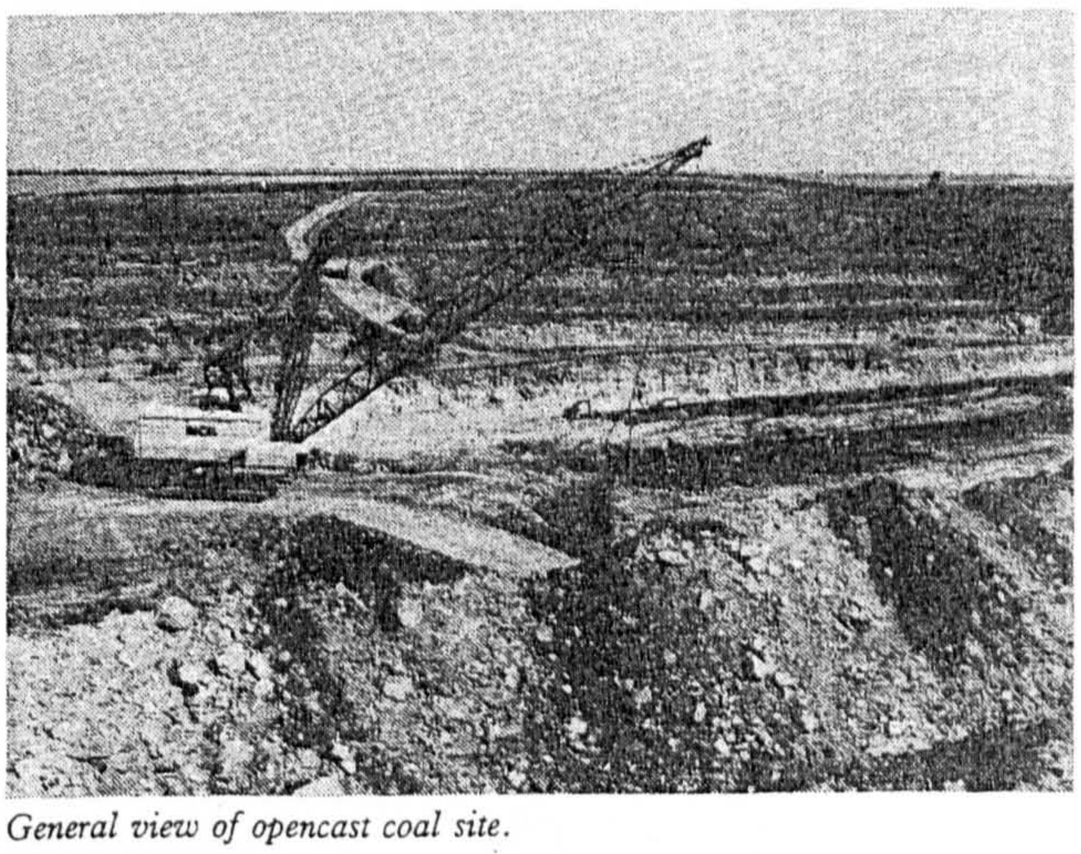
David Hughes

***David Hughes** works with the Opencast Executive of the NCB and in this article he deals with the important geotechnical aspects of opencast mining.*

Opencast coal production by the National Coal Board in England, Wales and Scotland is currently around 14 million tonnes per annum and is obtained from about 50 working sites. Also a small quantity of coal is produced from private licenced opencast sites.

The NCB Opencast Executive now covers England and Wales only, and is divided into five regions, with headquarters located at Mansfield. The NCB Scottish Area is responsible for opencast coal mining in Scotland.

Opencast coal sites are subject to minerals planning procedures before being "authorised" by the Secretary of State for the Environment. It is the Opencast Executive who prove the mineral quantities and feasibility of working the site and subsequently obtain the planning consent and authorisation. However, the mining operations involved in winning the coal and restoring the site are carried out by specialist civil engineering or opencast mining firms under contract. The



General view of opencast coal site.

Opencast Executive closely supervises these coal production contracts and controls the coal preparation and marketing functions.

For England and Wales only there are about 560 professional staff employed by the Executive and these include geologists, engineers (of various disciplines), land surveyors, lands

officers, planners, accountants and administrators. There are just four geotechnical engineers employed as such and their backgrounds include civil and mining engineering as well as geology.

Opencast Mining Operations

The simplified sequence of operations for single seam workings is shown in Figure 1. Topsoil and subsoil are removed by tractor scraper and placed in storage mounds on the site perimeter. Excavation to the coal seam in the initial cut is usually by face shovel and the spoil is transported by dump truck to the overburden mound for storage. Successive cuts are then excavated by dragline and/or face shovel with spoils going into the previous cuts. The final void is backfilled using the spoil from the overburden mound (i.e. from the initial cut) and subsoil and topsoil are replaced. Normally, agricultural restoration is carried out and is supervised and managed over a minimum five year period by the Ministry of Agriculture, Fisheries and Food (M.A.F.F.). Sometimes sites are restored for other purposes, e.g. forestry, recreation, or industrial development. Figure 1 shows that there is usually a nett volume increase in the backfill materials (bulkage) and this is accommodated in the final restoration contours.

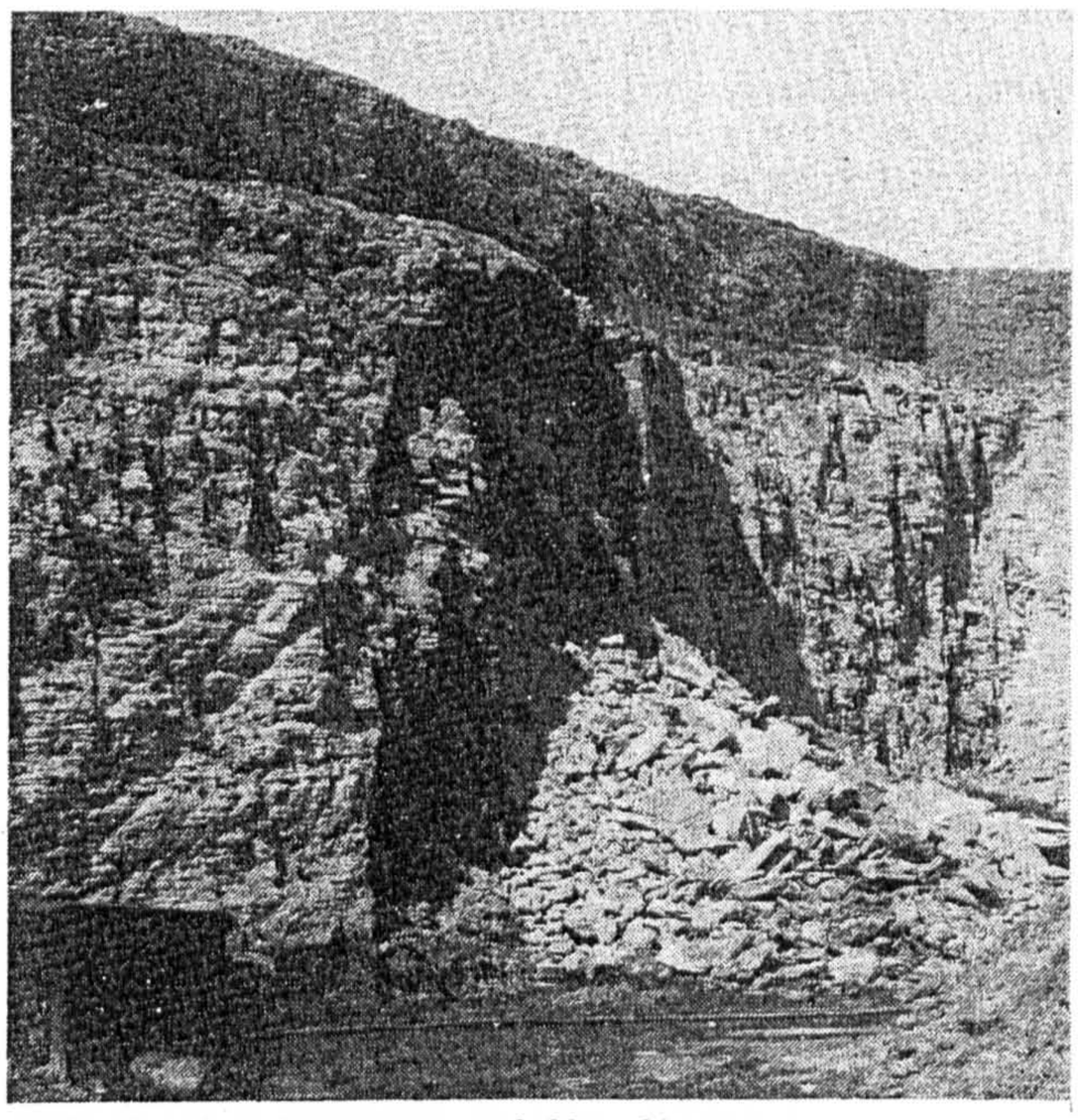
Figure 2 shows a plan and section of a typical multi-seam operation and illustrates some of the terminology used to describe the main features of an opencast mine. The geology greatly influences the way in which a site is operated e.g. where the strata are only very gently dipping the cuts may be aligned parallel to the strike, but in more steeply dipping strata cuts parallel to the dip direction are usually adopted to prevent the sliding of either loosewall or highwall into the working void.

In some situations the bedding dips are so steep that excavation plant and other vehicles cannot operate on the steep bedding surfaces. In these circumstances the Open Pit Method is adopted, which involves progressively lowering the floor of the pit and working the steep seams as the pit descends. This method necessitates storing a larger quantity of overburden spoil in mounds above ground level than in the more usual strike cut or dip cut methods. Replacement of the backfill is again progressive, the area being worked in a series of "panels" i.e. very wide cuts.

Ground Investigations

The geological conditions which exists at the shallow coalfields of Britain are very well known. The Coal Measures rocks are mostly overlain by drift deposits; occasionally Permo-Triassic formations are present; also minor igneous intrusions may be encountered.

Old underground workings exposed in opencast mining.



Highwall failure due to presence of old workings.

Openhole air-flush rotary drilling techniques are mainly used to prove the quantity of coal within a potential site, also to investigate the strata succession and depth of superficial deposits etc. The borehole spacing adopted is usually determined by the degree of structural complication and the occurrence of old underground workings. In a proportion of boreholes, core samples of the coal seams plus roof and floor measures are obtained for quality assessment and correlation. A smaller proportion of boreholes are core sampled for their full depth and logged in engineering terms.

Slimline (geophysical) logging of boreholes is used extensively to facilitate the interpretation of the various lithologies, coal seams, old workings and groundwater levels. New applications are being developed, particularly in relation to rock mass quality assessment.

Groundwater investigations mainly involve the installation of simple standpipes and piezometers to allow the recording of water levels over long periods. Packer tests are sometimes used to assess permeability. Also pumping tests are carried out occasionally.

Geotechnical investigations of the superficial deposits usually require the use of cable percussion boring techniques. In-situ testing and piezometer installation may be carried out; samples are obtained for soil mechanics laboratory testing.

The results of all these ground investigations are assessed and are presented in the form of the following documents:—

Geological/Prospecting:—
- Final Site Report
- Geological Plans
- Geological Sections
- Representative Coal Seam Sections
- Cored Borehole Sections
- Superficial Deposits Plans
- Old Working Plans

Geotechnical:—
- Groundwater Records
- Site Investigation (Superficial Deposits) Report
- Plans and Sections relating to the Siting of Spoil Mounds

Most of these documents are supplied to the firms tendering for opencast coaling contracts, either as contract documents or for information purposes.

Geotechnical Engineering and Contract Procedure

The stability aspects of working any particular site are assessed from a comprehensive study of all the ground investigation information. The Geotechnical Stability Report is prepared in which the stability of excavation faces, groundwater conditions and factors relating to the safety of adjacent structures and services are all discussed. In addition, a two part Code of Practice document is prepared which relates to the siting and construction of spoil mounds and is designed to ensure that the technical standards required by the Mines and Quarries (Tips) Act (1969) and Regulations (1971) are applied to opencast coal sites. Part 1 of the Code covers site investigation information and is completed by the Executive's competent person (geotechnical engineer), whereas Part 2 covers the design and construction of spoil mounds and is completed by a competent person nominated by the Quarry Manager (operating contractor's site agent).

When tenders are invited for the operation of an opencast site, each tenderer submits a Method Statement being a written description of how the site will be worked and is augmented by information shown on plans and sections. This information includes details of plant to be employed, sequence of operations, excavated slope profiles, tentative spoil mound locations, and dewatering schemes. Detailed scrutiny of all the Method Statement information is carried out by the Executive before the contract is awarded to ensure that the site will be operated in a satisfactory manner. The Code of Practice Part 2 procedure is only carried out by the successful tenderer.

Once the site is in operation the Quarry Manager is responsible for all matters relating to safety within the site. However, it is normal practice for the Executive's staff, and particularly the geotechnical engineers, to become involved in any significant stability problems that may arise.

Other Projects

Opencast mining often creates the need for associated engineering works, e.g. coal disposal points (crushing, storage and loading facilities), private haulroad systems; and diversions of public highways, major services and watercourses. Also the restoration of sites can lead to some interesting structures being required, especially if recreational or industrial afteruse is planned. The list of engineering works and structures is vast but a few examples include bridges, carriageways, railways, amenity lakes, pipelines, tunnels, retaining walls and storage structures. Specialist foundation processes include piling and grouting. Ground investigation and geotechnical engineering input is required for all of these projects.

In recent years the working and restoring of progressively larger opencast sites has lead to the identification of certain geotechnical engineering research needs. Various research projects have been undertaken in association with universities and government research organisations and the topics include physical properties of Coal Measures rocks, rock slope stability, excavation engineering, groundwater and settlement of restored opencast sites. Work is still continuing on several of these projects.

Acknowledgement

This article is published by kind permission of the N.C.B. Opencast Executive. The views expressed herein are entirely those of the author.

This article is based on an illustrated talk by ***David Hughes*** to the Northern Group of IG. We welcome articles based on IG Regional Group Meetings.

FIGURE 1
OPENCAST COAL MINING
Simplified Sequence of Working

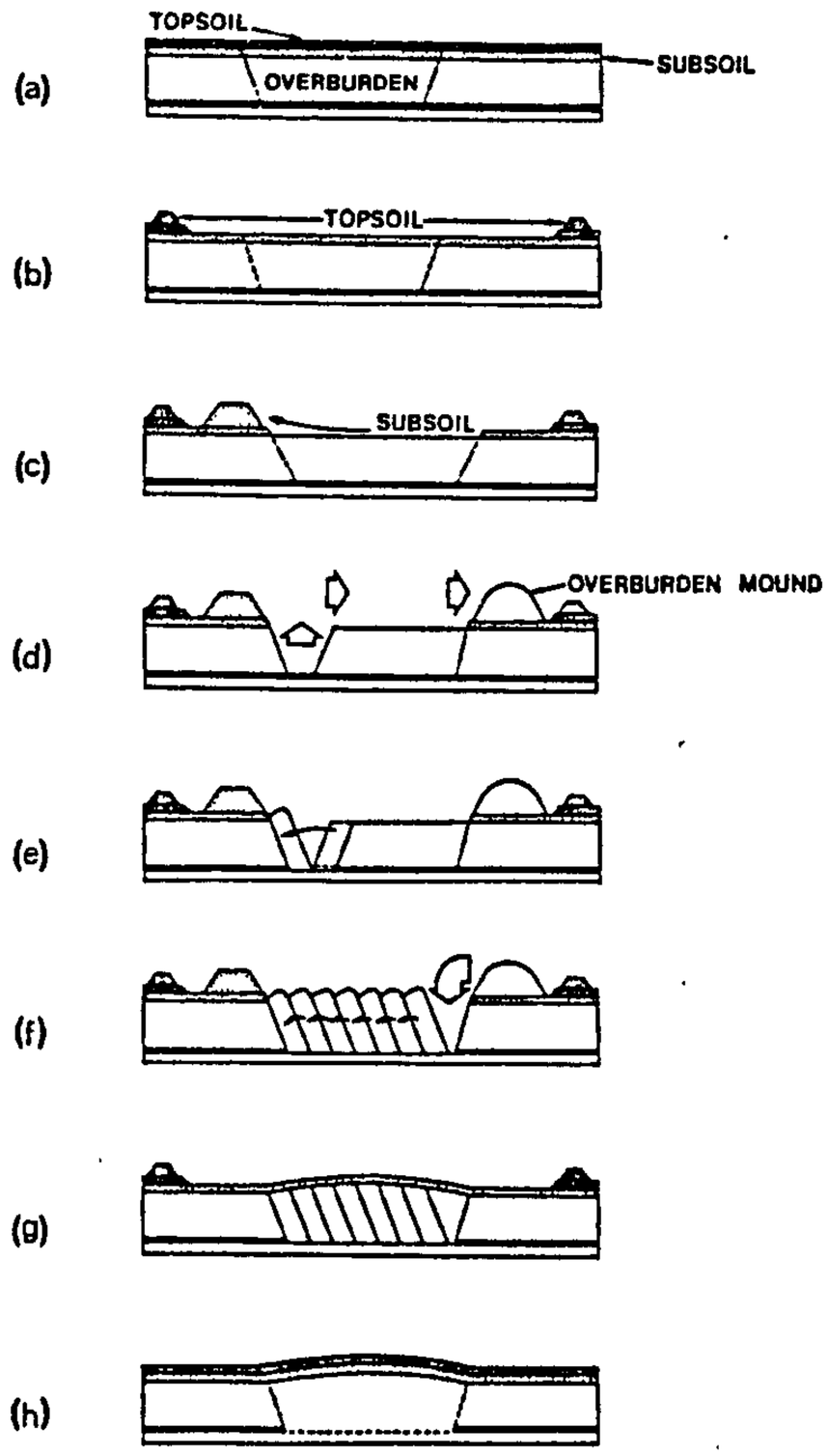
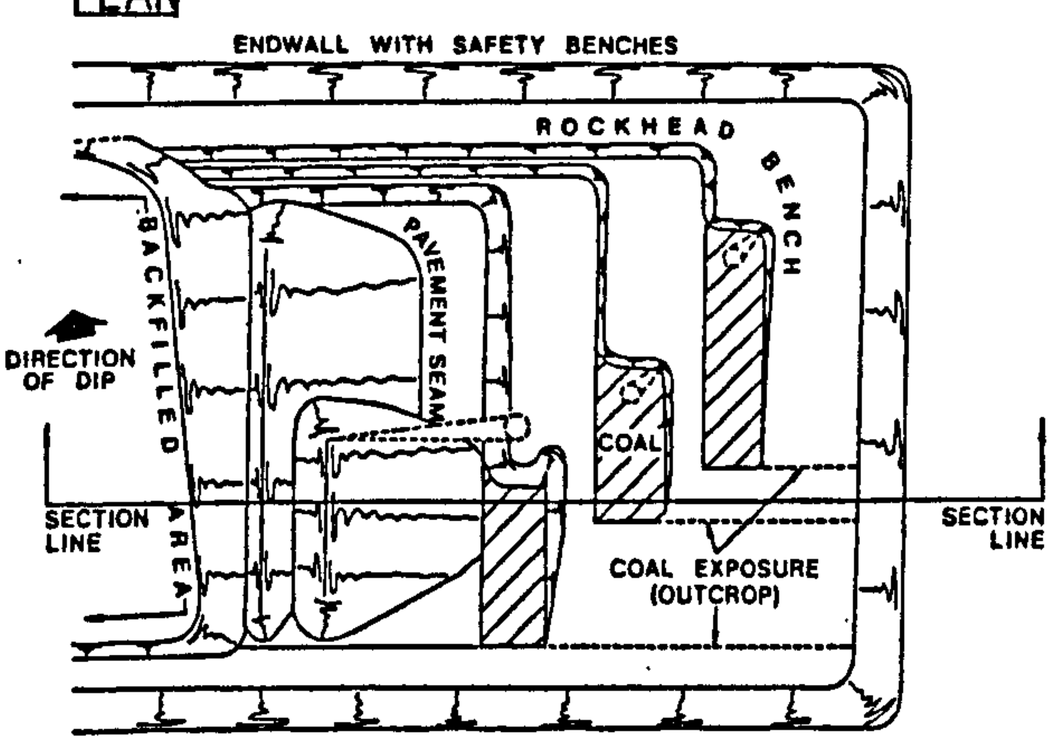
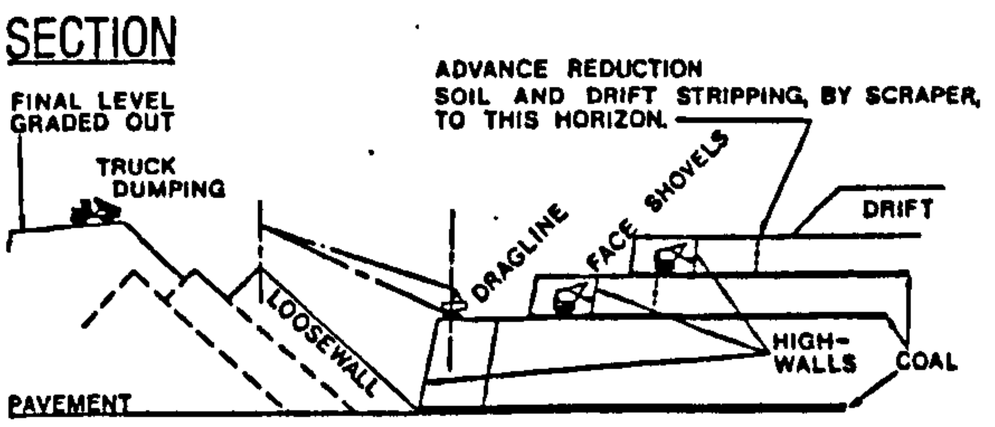


FIGURE 2
OPENCAST COAL MINING OPERATIONS





Figures 1 and 2 by courtesy N C B Opencast Executive.

PAPER C8 FAULTING – BUCKHEAD

Hughes, D.B. and McLean, A.A. 1986.

Stability monitoring of ground movements in faulted Coal Measures and Glacial Drift in Northern England.

Proceedings of the International Symposium on Geotechnical Stability in Surface Mining, Calgary, Alberta, November 1986, 355-363.

International Symposium on Geotechnical Stability in Surface Mining / Calgary / November 1986

Stability monitoring of ground movements in faulted coal measures and glacial drift, in north east England

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ABSTRACT

The Buckhead Opencast Coal Site is located in the Durham Coalfield of north east England and contains some 1.5 million tonnes of recoverable reserves. Work commenced on the Site in February 1980 and coaling operations are expected to cease in Spring 1987.

Area 'J' is one of fifteen separate coaling areas included in this opencast coal mining project. The main geological features are the Wigglesworth Fault with its many associated branch and splinter faults, steeply dipping bedding planes, old underground workings, groundwater and the variable nature and thickness of the glacial drift which overlies most of the Site. A minor public highway passes close to the Site boundary.

Excavation for two coal seams was carried out to a maximum depth of 35 metres and backfilling of the void followed closely behind the winning of the coal. Throughout the period of excavation and backfilling operations, ground movements were monitored using a variety of surface survey stations and some simple slip-indicators. Groundwater levels were also monitored using piezometers.

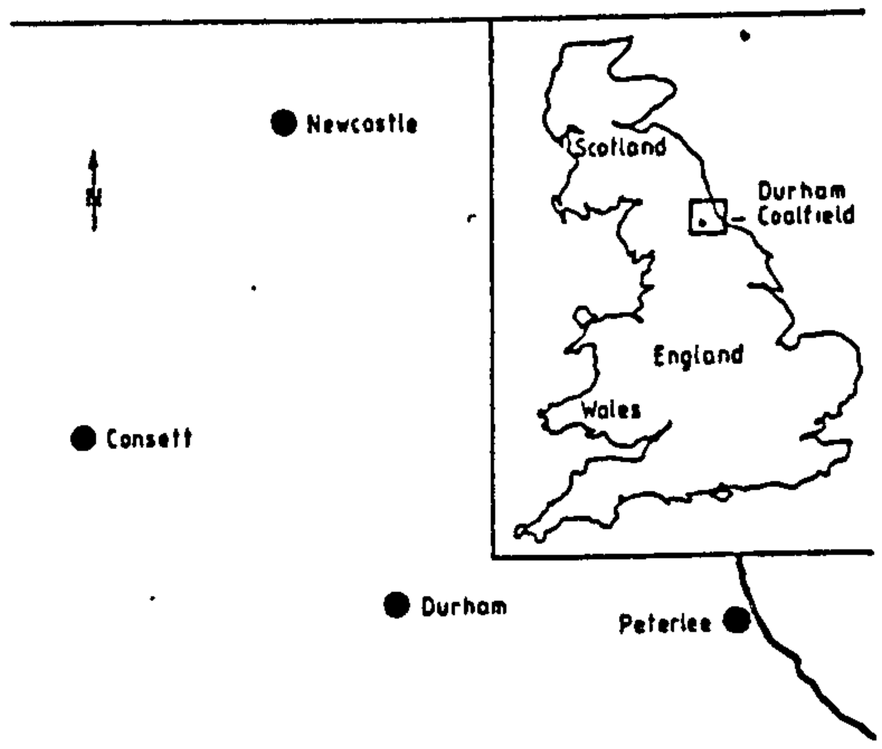
The paper includes an account of the ground investigations and survey data for this stability monitoring exercise, and discusses the working of the Site in relation to the results obtained.

INTRODUCTION

Situated in the Durham Coalfield in north east England some 30 miles to the south of the city of Newcastle upon Tyne (Figure 1), Buckhead Opencast Coal Site covers an area of around 317 hectares and is expected to yield approximately 1.5 million tonnes of coal when completed. Work commenced on the Site in February 1980, and coal production is expected to cease in the spring of 1987, with restoration work continuing for a further year at least. Virtually all of the land within the Site area was used for agriculture prior to opencasting, and will be returned as such following a five year programme of management by the Ministry of Agriculture Fisheries and Food (M.A.F.F.). on behalf of British Coal.

The Site is divided into two parts by a minor public highway known as Esperley Lane, and is further divided into fifteen separate coaling areas due to geological factors (Figure 2). This paper is concerned with the excavation and stability conditions in the north eastern part of Area 'J' where it adjoins Esperley Lane.

Opencast Coal Sites in the United Kingdom, and particularly in the Durham Coalfield tend to be much smaller operations that those in North America and elsewhere in the world. A brief account of the size of U.K. operations, together with descriptions of working methods and details of some geotechnical and stability aspects is given by Hughes and Leigh (1985). A more detailed account of operations at Buckhead Site is given by McGregor and Brophy (1982).



Linear Scale	Location Plan of Buckhead Site	Figure 1

GROUND CONDITIONS

Buckhead Site is overlain by superficial deposits of glacial drift materials of varying thickness, and is located near the south western extremity of the Durham coalfield. There are twelve named coal seams within the Site varying between 0.2m and 2.0m in thickness, with some seams dividing into two leaves. The major geological feature is the Wigglesworth Fault which mainly occurs within the northern part of the Site, and in the area east of Esperley Lane where it divides into several branch and splinter faults. Steeply dipping bedding planes and old "board and pillar" underground workings are also common features throughout the Site.

In the north eastern part of Area 'J', over a length of about 400m, a branch of the Wigglesworth Fault strikes almost parallel with the contract excavation limits and the public highway. The depth of excavation along this working boundary was around 30m to 35 m. The two coal seams which were taken at this location were the Main and Maudlin seams, both around 1.5m to 2.0m in thickness, and estimated to have 65% and 40% old workings voids respectively. The dip of the seams was about 1 in 4 north eastwards (Figure 3).

The geological structure interpreted from the forty or so boreholes which were drilled in this vicinity are shown in geological sections A-B, C-D and E-F. (Figures 4(1), (2) and (3)) respectively. It can be seen that the Wigglesworth Fault is a wide fracture zone containing several displacement planes which vary in dip, strike, throw and lateral continuity, (it was thought that the geological structure was rather more complicated than it was possible to show on the geological sections). In addition there are old workings in the coal seams immediately adjacent to both sides of the fault zone, including beneath the highway. The boreholes also showed the Coal Measures bedrock to be overlain by between 5m and 15m thickness of glacial drift deposits comprising stiff boulder clays and containing pockets or lenses of water-bearing sands and gravels.

MONITORING INSTALLATIONS

Because the structural geology was found to be so complex and variable, it was considered that the results of stability calculations based on simplifying assumptions were probably quite unrealistic in these conditions. Instead, the safety stand-off margins from the highway boundary to the permitted (contractual) excavation limits and the batter profile allowances,were all based upon previous experience gained from working through or adjacent to other complex fault zones. However, in order to give an early warning of any major instability of the sides of the excavation, and hence ensure the safety of the Site boundary and public highway, it was decided to install a ground movement monitoring scheme.

As shown in Figures 3 and 4 (1) (2) (3), three sections (A-B, C-D, E-F) each about 50m apart and aligned nearly normal to the face of the excavation, were chosen for initial ground monitoring purposes. For simplicity of reading and interpretation by Site staff, simple slip-indicators (slip-rods) were chosen for below surface monitoring, and the construction details for these are shown in Figure 5 (1). These are simple and

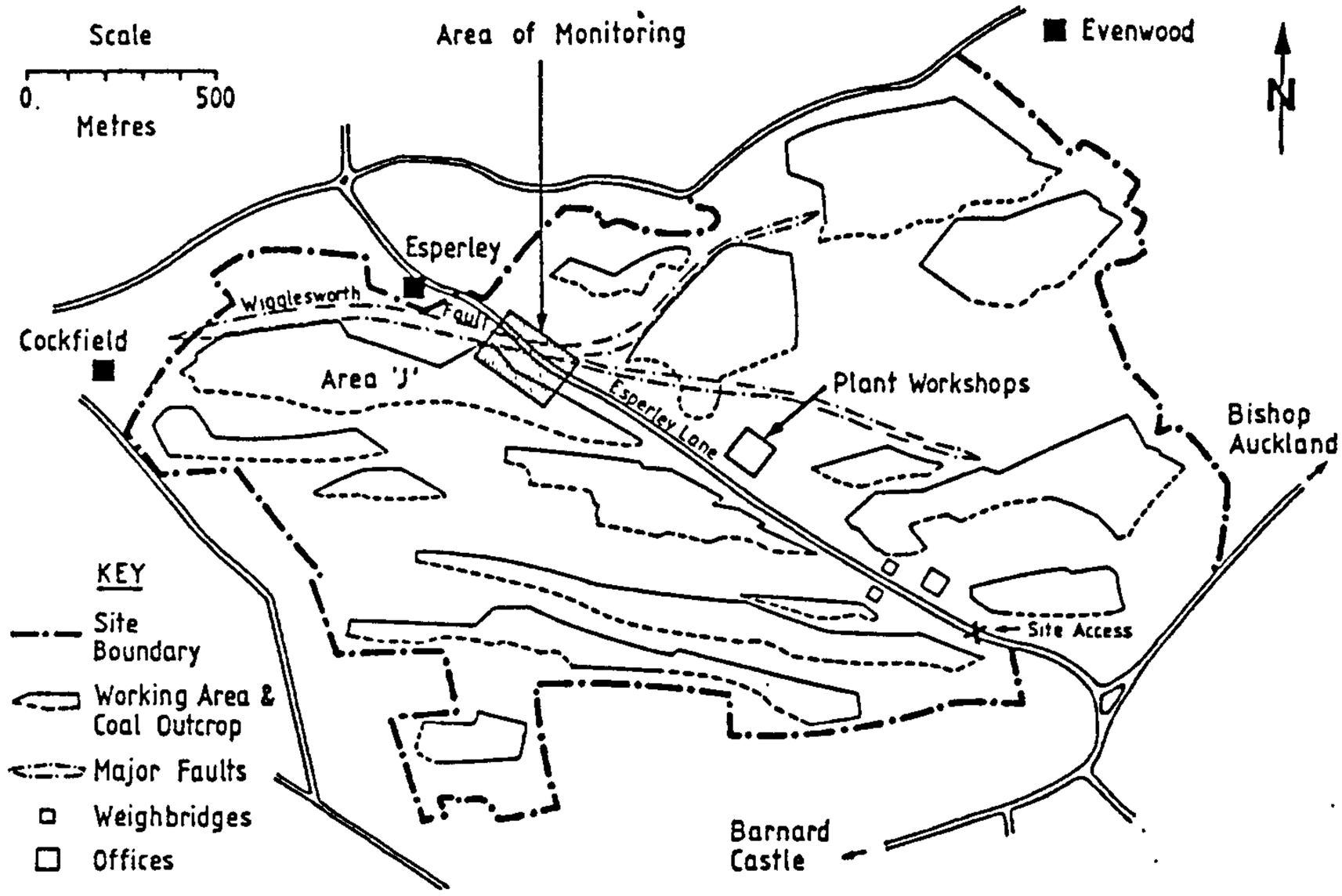


Fig. 2 Buckhead Site Plan

easy to use devices for locating planes of differential movement (slip-surfaces) and involve the use of metal rods of various lengths (slip-rods) being raised or lowered through plastic tubes. The formation of a restriction to the free passage of the slip-rods identifies a possible slip-surface. A total of six slip-indicators (i.e. two on each section) were installed each to a depth near or below the full depth of the excavation, and four of these also doubled as simple piezometers.

In addition ten surface survey stations (Stns) were installed in the positions shown on Figure 3 and to the construction details shown in Figure 5(2). These were numbered 6 to 15 inclusive (Stns 1 to 5 had previously been installed in a different part of the Site) and were surveyed for vertical movement using a Wild precise level and invar levelling staff, and for horizontal movement using a Wild Tachymat total station. During the early stages of the excavation works Stns 6 and 7 were damaged by site plant, and were replaced by Stns A B and C which were set further away from the crest of the excavation slope.

Later on during the period when excavating through this area, tension crack marker pegs, road surface studs, and additional surface survey stations (Stns 16 to 22) were also installed.

METHOD OF WORKING

As stated earlier, steep bedding plane dips are a common feature at Buckhead Site. Hence, much of the overburden has had to be excavated by face-shovel and transported by dump trucks, with dragline 'dig and cast' operations only being feasible in a few of the fifteen coaling areas.

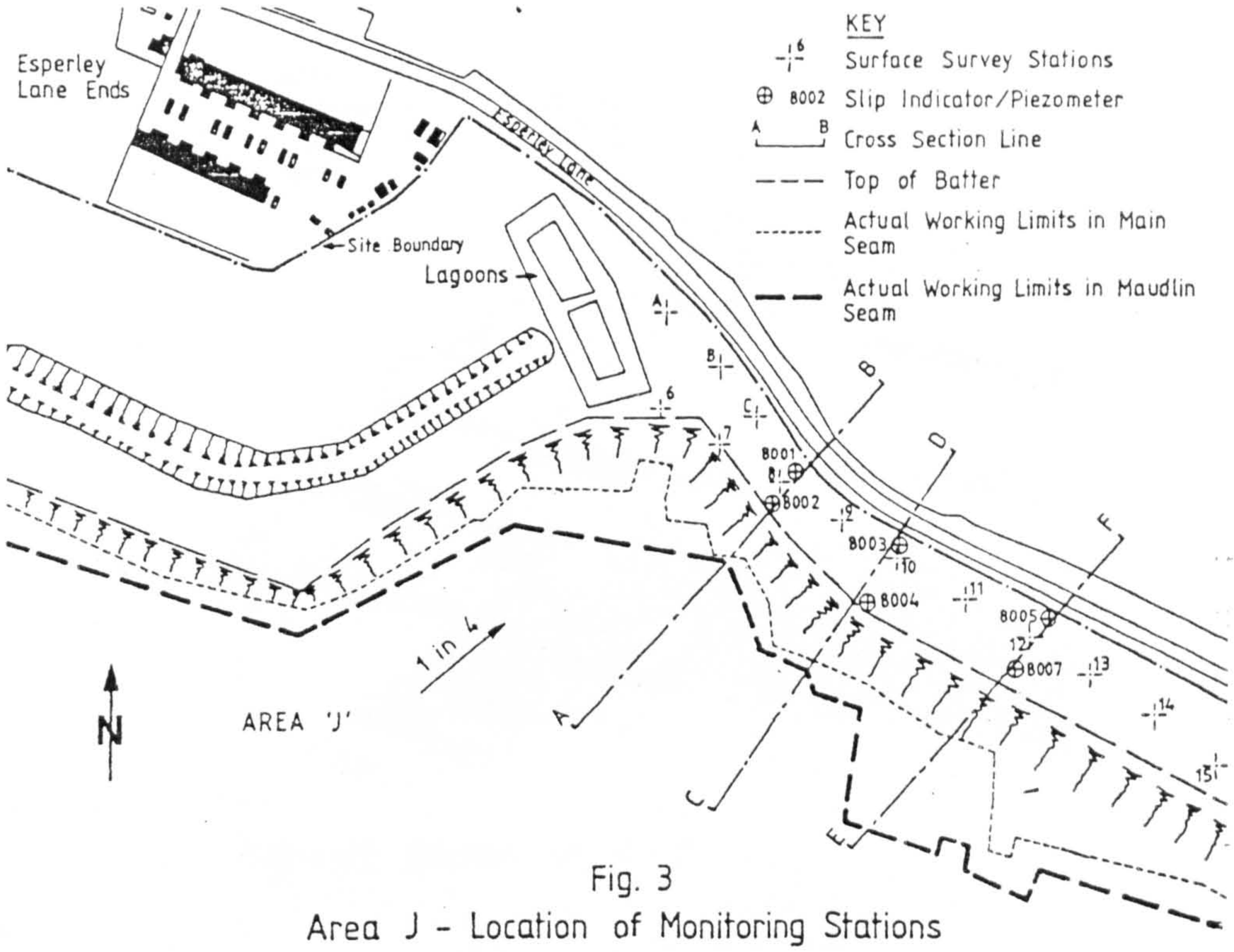


Fig. 3
Area J - Location of Monitoring Stations
Plus Coaling Areas

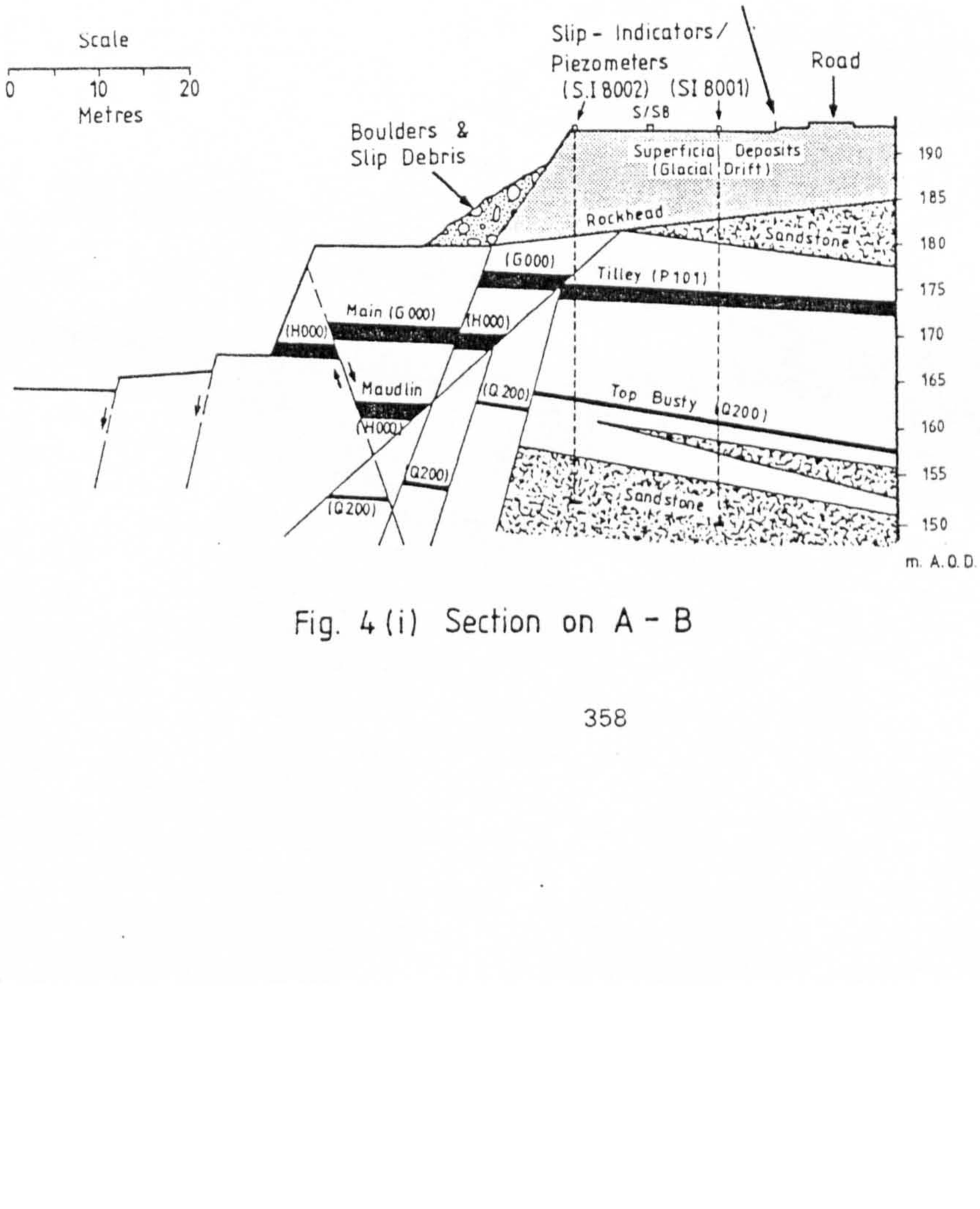


Fig. 4 (i) Section on A - B

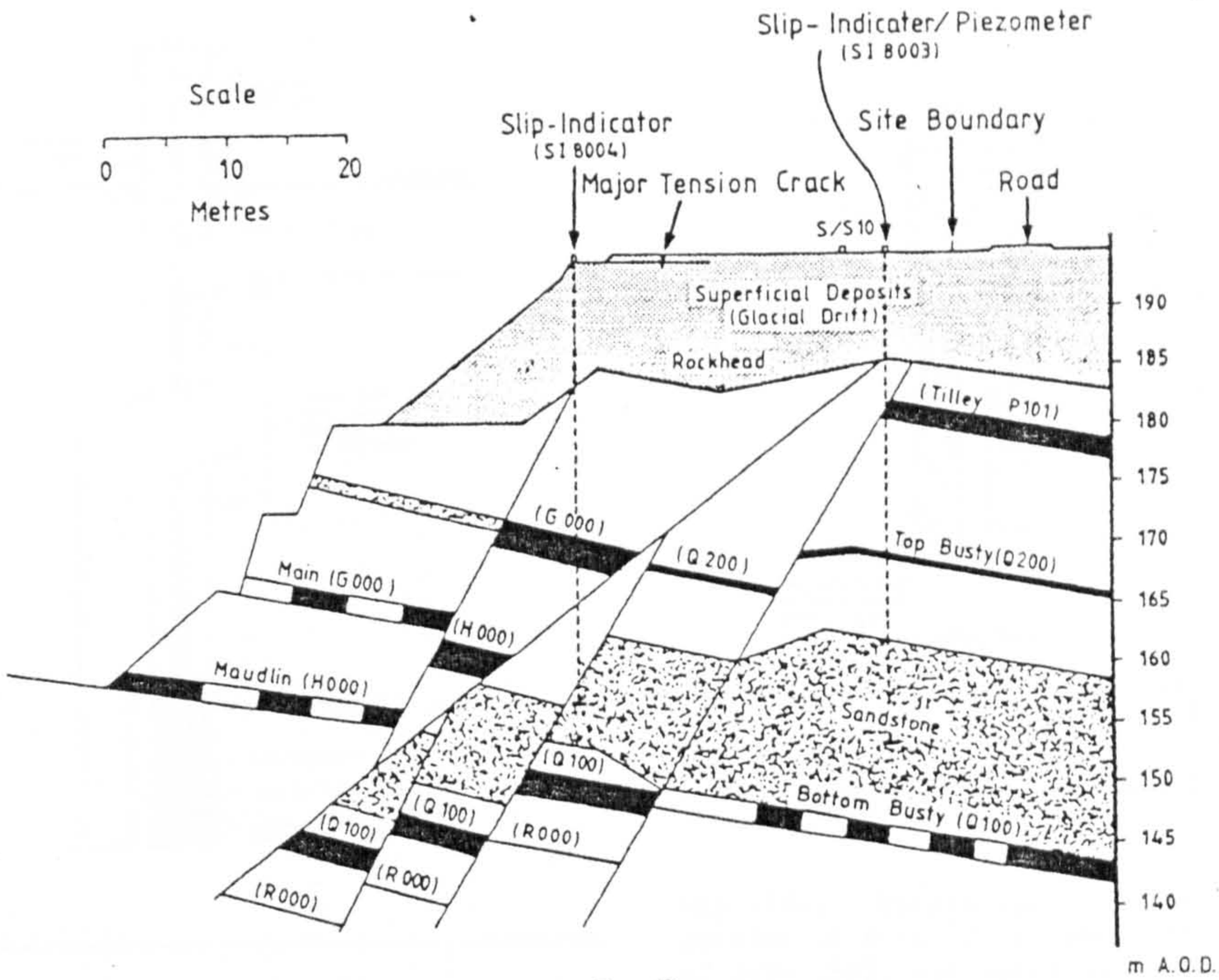


Fig. 4 (2) Section on C - D

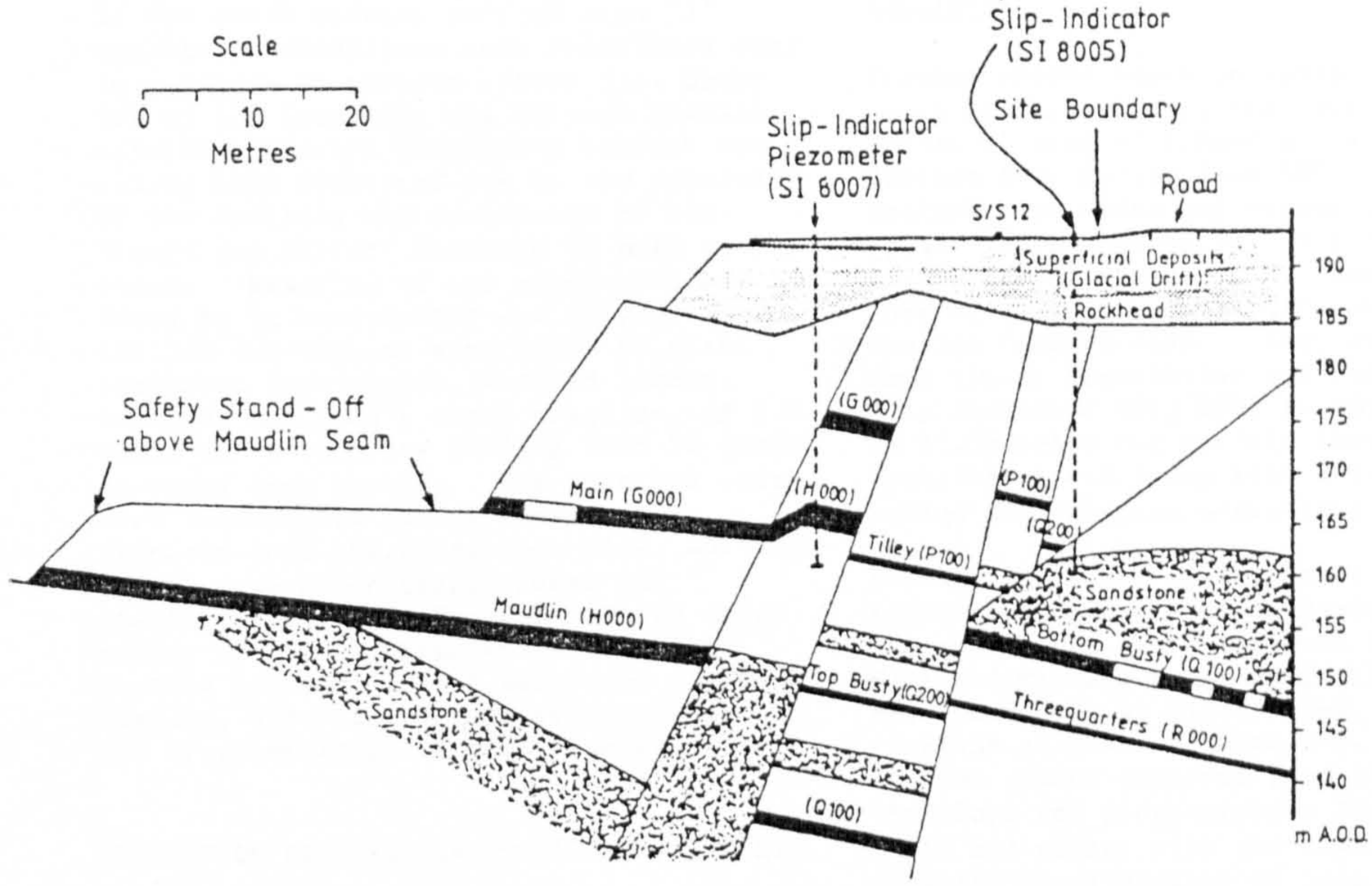
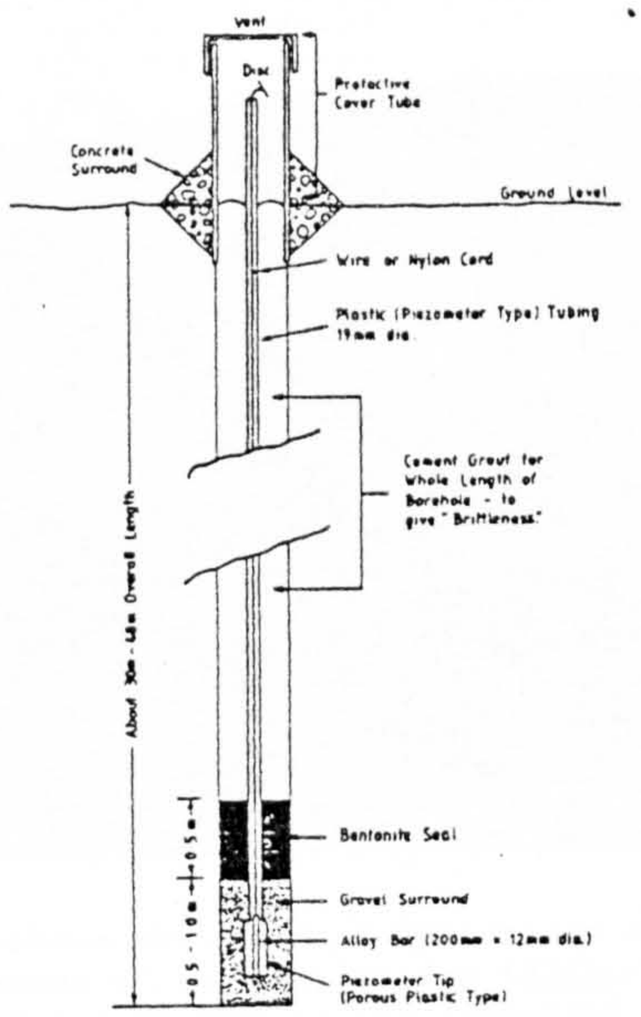
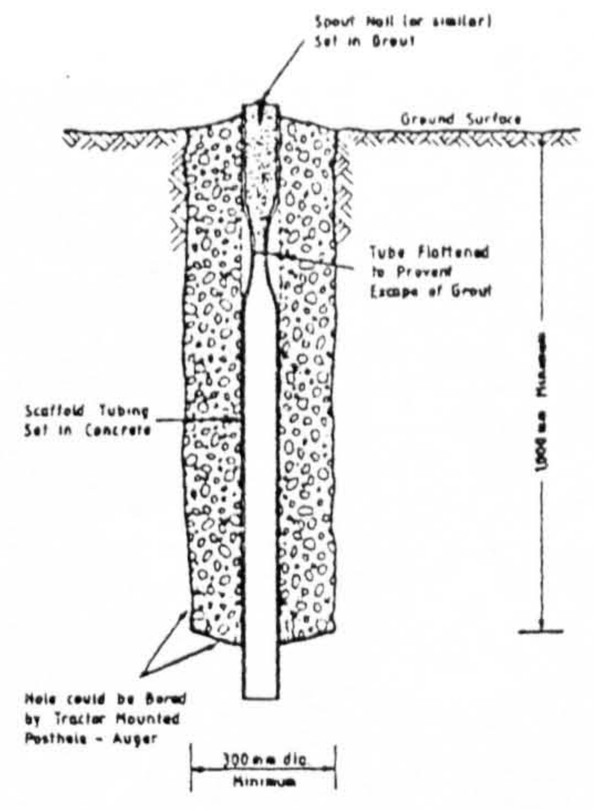


Fig. 4 (3) Section on E - F



Borehole Slip Indicator / Piezometer | Figure 5(1)



Surface Survey Station | Figure 5(2)

In the north eastern part of Area 'J' excavation conditions were relatively easy in relation to machine effort i.e. about 30% of the total dig was through glacial materials and the underlying bedrock was fairly well fractured due to the proximity of the faulting and occurrence of old 'board and pillar' workings in both coal seams. Blasting of the overburden was found to be unnecessary and the work was carried out with an assortment of plant including face-shovel, wheeled loader, backhoe, and even a small dragline, of 5.4 cubic metre capacity loading into 50 tonne capacity dump trucks. The problems which were encountered during excavation resulted from the steep seam dips, working across many minor fault planes and displacements, and cleaning out the large number of old workings voids. A small backhoe and face-shovel were used for coal winning, and road lorries were employed for transportation of coal from the Site.

EXCAVATION PROGRESS AND MONITORING RESULTS

Monitoring of the surface stations adjacent to Esperley Lane commenced in November 1982, and continued through to May 1984. Excavation work reached this section of Area 'J' at about the beginning of July 1983, and backfilling was substantially completed by March 1984. Hence the monitoring excercise covered the period from 8 months before the excavation reached the critical location until some 2 months after backfilling had been completed.

Tension cracks began to appear near the crest of the slope in the north east corner of Area'J' (immediately north of Section A-B) during July 1983, and continued to widen and extend until a planar failure occurred (on a fault plane) at the end of August 1983, causing the loss of a small quantity of coal from the Maudlin (lower) seam. Also about this same time a restriction was recorded in Slip Indicator (SI) 8002 in the range 21m to 23.5m below the surface (bts). Backfilling was being kept fairly close behind coal winning operations (Figure 6).

Through the month of September 1983, the excavation advanced south eastwards and by the end of the month was down to both the Main and Maudlin seams at Section A-B, the Main seam only at Section C-D, and rockhead only at Section E-F. Several tension cracks appeared near the crest of the slope and progressively increased in width and number with the advancing excavation and passage of time. These cracks appeared to be associated with the initial formation of quasi-circular failures in the glacial drift materials.

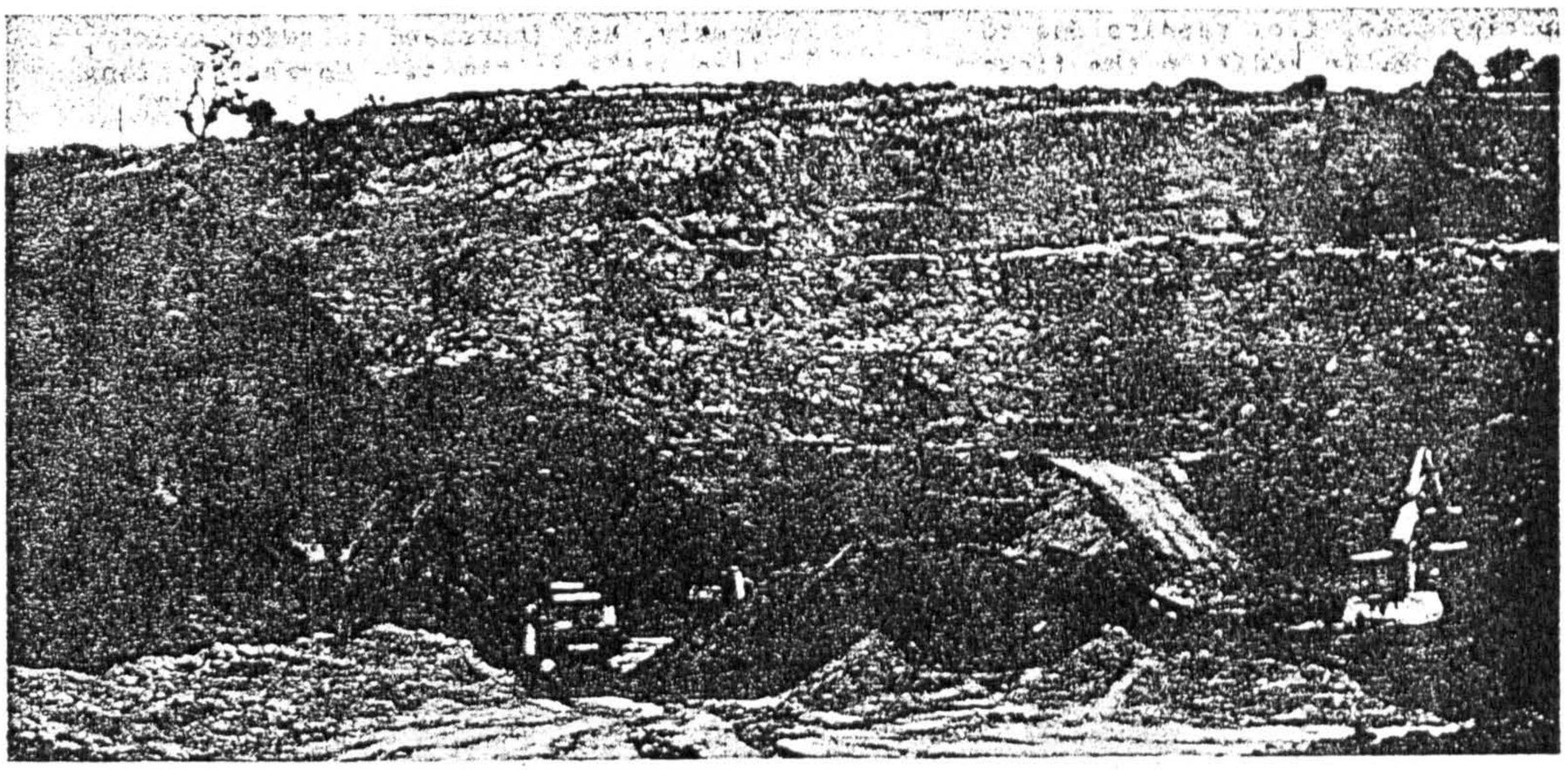
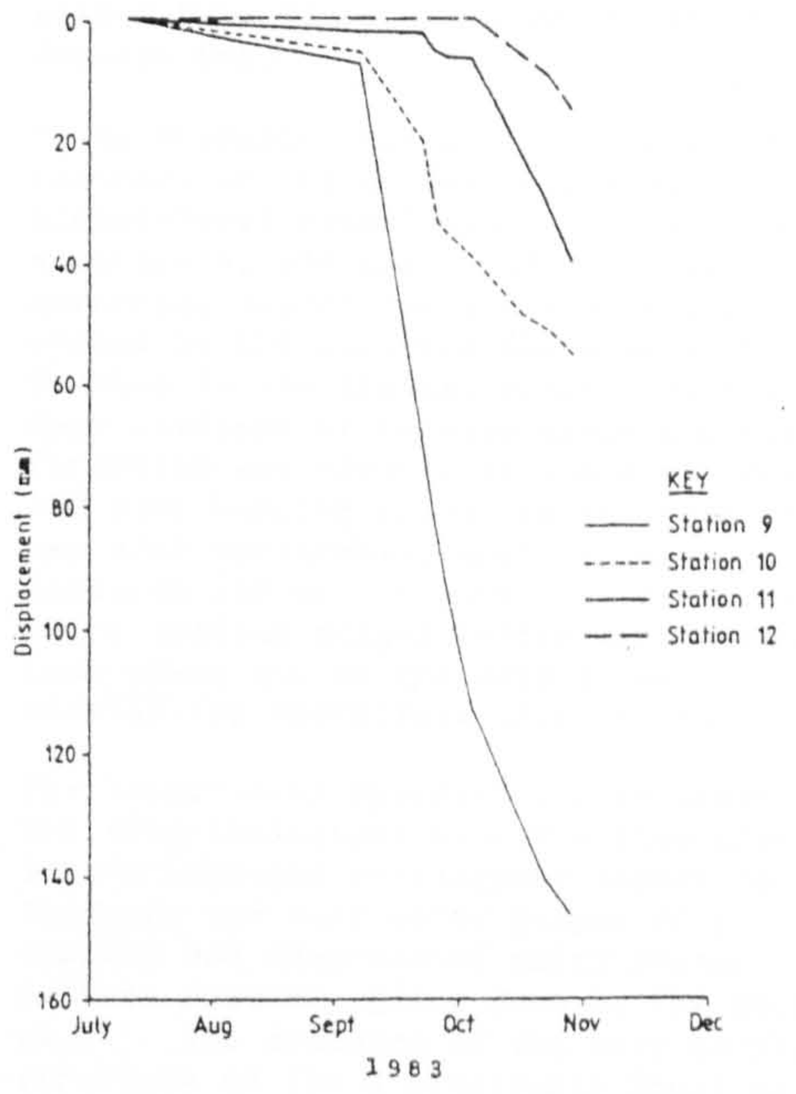


Figure 6 shows the state of Operations at Section C-D (looking eastwards) on 30.9.83. The closeness of the backfilling (darker material) and cracks in the side of the excavation can be seen on the left-hand side. A failure in the superficial deposits is shown on the right-hand side and Esperley Lane beyond, is indicated by the hedges, trees and telegraph poles silhouetted against the sky-line.

In mid-September 1983, the commencement of large downward movements were recorded on Stns 9 and 10 and on the covers of SI 8002 and SI 8004, and by the end of that month these displacements had increased to 112mm, 39mm, 172mm and 250mm respectively. Hair line cracks had begun to appear in the surface of Esperley Lane near Section C-D, and road studs were installed to facilitate monitoring of these. Also during the period mid-September to mid-October restrictions were recorded in SI 8001 at 11m and 19m bts, in SI 8003 at 15.5m and 24.5m bts, and in SI 8004 at 5.5m and 11.5m to 12.6m bts.

In order to arrest further ground movements a stand-off to the contractual coaling limits was decided upon, as shown in Figure 3. Also backfilling was brought as close as possible to the coaling operations to help to buttress the side of the excavation (Figure 6).

By the end of October 1983, downward movement at Stns 9 and 10 and SI's 8002 and 8004 had increased to 148mm, 56mm, 219mm and 975mm respectively (Figure 7). However, no further movements or cracking had been recorded in Esperley Lane. Repairs to the small area of damaged carriageway surface were carried out by the Highway Authority as part of a patching operation covering the whole of



Displacement of Survey Stations — Figure 7

Esperley Lane, i.e. repairs due to trafficking. In addition the first downward displacements were recorded for Stns 11 and 12 and SI's 8003, 8005 and 8007; being 41mm, 16mm, 36mm, 10mm and 34mm respectively. SI 8001 suffered machine impact damage at about this time, but no significant vertical movement of the cover had been recorded prior to this (although restrictions to the slip-rods had been recorded in mid-October, see above).

As excavation and coaling operations continued south eastwards during the period November and December 1983, downward displacements at most surface stations only increased by a few millimetres. During the month of January 1984, Stns 13, 14 and 15 (located south east of Section E-F) showed some minor downward movement and by mid-February 1984, this had increased to 38mm, 52mm and 45mm respectively. Backfilling was still being kept very close to the excavation and coaling operations, but some further very minor cracks appeared in the surface of Esperley Lane between Sections C-D and E-F, and these were later repaired by the Highway Authority.

Extra surface stations (Stns 16 to 22) were also installed in November 1983 on a line continuing south eastwards from Stn 15, but no significant movements were recorded on these prior to completion of backfilling (March 1984) and cessation of monitoring (May 1984). However some planar and toppling failures did occur in this final section of excavation in Area 'J' due to the presence of old workings in the Main seam and the collapse of the overlying sandstone strata.

Vertical displacement monitoring (levelling) of surface stations had been carried out mostly at weekly (but occasionally twice weekly) intervals during the excavation period. Horizontal displacement monitoring was performed much less frequently because the Tachymat total station method did not give such accurate results as the precise levelling which was used for vertical measurements. However horizontal movements of up to 90mm were recorded during September and October 1983 and tend to coincide with the large vertical movements also recorded at that time.

Checking of slip-indicators and piezometers was generally carried out weekly, but increased to twice weekly, and even daily at times. No restrictions were recorded in SI 8005 or SI 8007 (Section E-F) at any time. Piezometers were generally either dry or recorded water levels very close to their base, i.e. close to the base of excavation level (Maudlin seam), and the maximum recorded water level rise was only about 6m. Where water level rises did occur they did not relate to records of surface movement or to records of restriction in the slip-indicators, but rather they coincided with periods of higher rainfall (e.g. February 1983 which was before any excavation had started at this location).

DISCUSSION AND CONCLUSIONS

A variety of relatively minor slope failures occurred during the working of this particular section of Area 'J', including a planar failure in bedrock in the north east corner (north of Section A-B), the initial stages of several quasi-circular failures in the glacial drift (between Section A-B and Section E-F), and planar/toppling failures in the sandstone overlying the old workings in the Main seam (about 50m to 150m south east of Section E-F).

It is probable that most of the movements recorded on the surface stations, the higher-level restrictions in the slip-indicators, and the tension cracks occurring nearer the slope crest, were caused by the failures which began to develop in the glacial drift. Although much evidence of tension crack and graben formation was visible at the slope crest, and some bulging above the rockhead bench was also noticeable, most of these failures did not develop very far beyond their initial stages before buttressing took place due to the very close backfilling operations (Figure 6).

The lower-level restrictions recorded in the slip-indicators plus the fine cracks in the adjacent carriageway appear to indicate the very early stages of a complex and deep-seated multi-planar failure possibly going down to the Maudlin seam. The evidence of the very complex structure of the Wigglesworth Fault zone, plus the use of only a small number of slip-indicators, discouraged any attempt at a rigorous stability analysis, as the shape and limits of such a failure could not be defined with any certainty.

Where geologically complex structures coupled with steep bedding planes occur near to public utilities and where usual stability analysis methods are impracticable, coaling limits have to be determined with regard to maximising coal recovery but at the same time attempting to ensure the stability of the Site boundary.

However, by installing this simple low cost monitoring system of surface survey stations and slip-indicators, and by carrying out regular readings and checks, early warnings of the formation of a large deep-seated failure were obtained. This led to the modification of excavation operations by standing-off from the contractual coaling limits so reducing the overall slope gradient and leaving extra dead-weight at the toe, and to acceleration of backfilling operations to effect buttressing of the slope. Hence ground movements close to the Site boundary were greatly retarded and only very minor damage occurred to the highway.

The quantity of contractual coal which was not recovered due to this localised stability problem was only about 5000 tonnes.

ACKNOWLEDGEMENTS

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PAPER C9 DRILLEX '87

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Sub-surface investigations by drilling and boring for opencast coal mining projects in the United Kingdom.

Proceedings of DRILLEX '87 Conference, Stoneleigh, Warwickshire, April 1987, 57-64.

Subsurface investigations by drilling and boring for opencast coal-mining projects in the United Kingdom

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SYNOPSIS

Opencast coal production in the U.K. is currently around 14 million tonnes per annum, and requires a large turn-over of sites. The Coal Measures strata from which the product has to be won have often been subjected to earlier mining by underground methods; also structural features such as faulting and steeply inclined bedding are common. Many sites are covered by drift deposits, mostly of glacial origin.

Openhole air-flush rotary drilling techniques are mainly used to prove the quantity of coal within a potential site, and to investigate the strata succession and depth of superficial deposits. The borehole spacing adopted is usually determined by the degree of structural complication and the occurrence of underground workings. Core samples of the coal seams plus roof and floor measures are obtained from a proportion of boreholes for quality assessment and correlation. A smaller proportion of boreholes are core-sampled for their full depth and logged in engineering terms, as an aid to both stability and diggability assessment. Geophysical (slim-line) logging techniques have been developed which are used to provide information on coal seam and rock mass characteristics. Investigations in the superficial deposits usually require the use of cable percussion boring techniques, and samples are obtained for soil mechanics laboratory testing. Various types of instruments are installed in boreholes in order to monitor groundwater conditions and ground movements.

INTRODUCTION

The annual production of coal by opencast methods in the United Kingdom is currently about 14 million tonnes, and is mainly won from British Coal sites, of which there are around fifty located in England, Scotland and Wales. In addition a relatively small proportion of coal is provided from private licensed opencast sites.

The extraction of 1 tonne of coal may typically involve the removal of around 10 to 20 cubic metres of overburden. Average working depths are around 40m, with 100m depth being quite common, and very exceptionally seams over 200m below the surface being taken (e.g. Westfield - Scotland). The types of excavation plant used include tractor-scrapers, face-shovels and draglines. Blasting or ripping is often employed to facilitate the digging of massively bedded strata. Site areas range between 30 and 700 hectares, this being dependent upon the depth and quantity of coal available and on the amount of temporary spoil storage (overburden, subsoil and topsoil) that is required.

Proposed opencast coal sites are subject to minerals planning procedures prior to being "authorised" by the Secretary of State for the Environment. It is British Coal who prove the mineral quantities and feasibility of working the site, and subsequently obtain the planning consent and authorisation. However, the mining operations involved in winning the coal and restoring the site are carried out by experienced civil engineering or opencast mining firms under contract. Marketing functions remain the responsibility of British Coal.

For all potential opencast coal projects a large amount of ground investigation work is necessary in order to prove that the site can be worked both economically and safely. These investigations are usually considered under two headings:-

(i) PROSPECTING - to prove the quantity and quality of coal available; also to prove the geological structure and strata succession.

(ii) GEOTECHNICAL - to investigate all the factors which may affect the stability of excavations and spoil mounds, and to investigate the nature of the strata in relation to its excavation (diggability) characteristics.

Much of the information relating to geological structure and strata succession, which is gained from normal prospecting activities, is basic information onto which geotechnical data and parameters are added. Therefore geotechnical investigations are mostly carried out subsequent to prospecting work, and as an extension thereof.

The extent of the ground investigations at any particular site are determined by the physical and geological conditions. Experience in the north east of England indicates that for a typical site containing around 2 million tonnes of recoverable coal the prospecting stage may involve between 1000 and 2000 rotary boreholes. Geotechnical boreholes may number between 50 and 100 and may include cored boreholes for engineering logging, instrumented boreholes, and cable percussion boreholes in superficial deposits.

GEOLOGICAL CONDITIONS

The winning of opencast coal in the U.K. involves excavating mainly through Coal Measures strata. Other sedimentary or igneous formations may be encountered, and sites are nearly always overlain by superficial deposits. Groundwater is often present within the strata.

Superficial Deposits

Most coalfields in Britain have some superficial soil cover over the bedrock, but the depth and nature of these superficial materials varies considerably. In the north of England about a quarter of all the ground excavated on opencast sites comprises superficial deposits and mainly consist of glacial till (boulder clay, laminated clay, sand and gravel, etc.); also residual soils, alluvial deposits, peat, and made ground (colliery spoils and previous opencast backfill, etc.) commonly occur. Glacial till is usually present at opencast sites in South Wales, and in Scotland (where peat deposits are also frequently encountered). In the Midlands and Yorkshire there is only a very occasional cover of glacial material, but there are sometimes areas of residual soils, alluvium, and made ground.

Coal Measures Strata

Coal measures strata consist of interbedded sandstones, siltstones, silty mudstones, mudstones, coals and seatearths. Rythmic patterns of deposition are sometimes observed, though generally the picture is more complex and there are significant lateral as well as vertical changes. Due to the varying degree of competence of the strata, the stability of excavation walls in an opencast mine is governed by the rock types present and the joint or fracture pattern within them. The strata has been subject to faulting, shearing and thrusting in some areas. In addition, the presence of intraformational shear zones may have a direct bearing on excavation face stabilities. Collapse structures of broken and fragmented rock above former underground workings are also of great significance.

Other Strata

In some areas the Coal Measures have been intruded by sills and dykes of doleritic material (e.g. Acklington Dyke, Northumberland). In Durham and the East Midlands the subcrop of Permian strata consisting of poorly cemented sands and limestones is very occasionally encountered. In Cumbria this strata consists largely of sandstones; whereas in parts of the Midlands Triassic marls, sandstones and conglomerates may be present.

Groundwater

Groundwater is present within the bedrock at many sites, and may have a significant bearing on both the working method and stability aspects. Sandstone beds and coal seams containing old workings usually act as aquifers, whereas mudstones and seatearths may be aquicludes. Also, local "perched" water tables may occur within the

superficial materials, e.g. in old opencast backfill, or in sand and gravel lenses within the glacial drift.

PROSPECTING METHODS

The main requirements of any ground investigation exercise for a potential opencast site are to prove the quantity and quality of coal which exists there, also that the geological structure be established in some detail. The acquisition of this information permits an assessment of the economics of working the site to be made. The most important parameter to establish is the working ratio, i.e. the ratio of volume of recoverable coal to total overburden excavation, as this has the greatest effect on the unit cost of the product.

Before drilling commences on site information about ground conditions and coal reserves is obtained from the following sources:-

- Abandoned mine plans
- Plans of active underground workings
- Aerial photographs
- Reports and records of Government Departments (including British Geological Survey) and papers in learned journals
- Geological details from accessible rock exposures

Drilling Methods

All drilling is undertaken by independent drilling contractors and is usually carried out using rotary air flush drilling rigs which may be single tractor (Figure 1) or track mounted units with an integral or separate compressor unit. Boreholes are drilled vertically using standard tricone rockbits to a diameter of 98mm or 108mm, to depths of up to 200 metres.

Where possible the driller "reads" the hole using rock chippings brought to the surface by the air or air/groundwater flow and records a borehole log. However, the drilling of deep boreholes and the occurrence of old underground workings cavities invariably leads to "blind" drilling where no chippings are received at the surface. Information about the strata in these sections of the boreholes is obtained from geophysical logs which are referred to later in this paper.

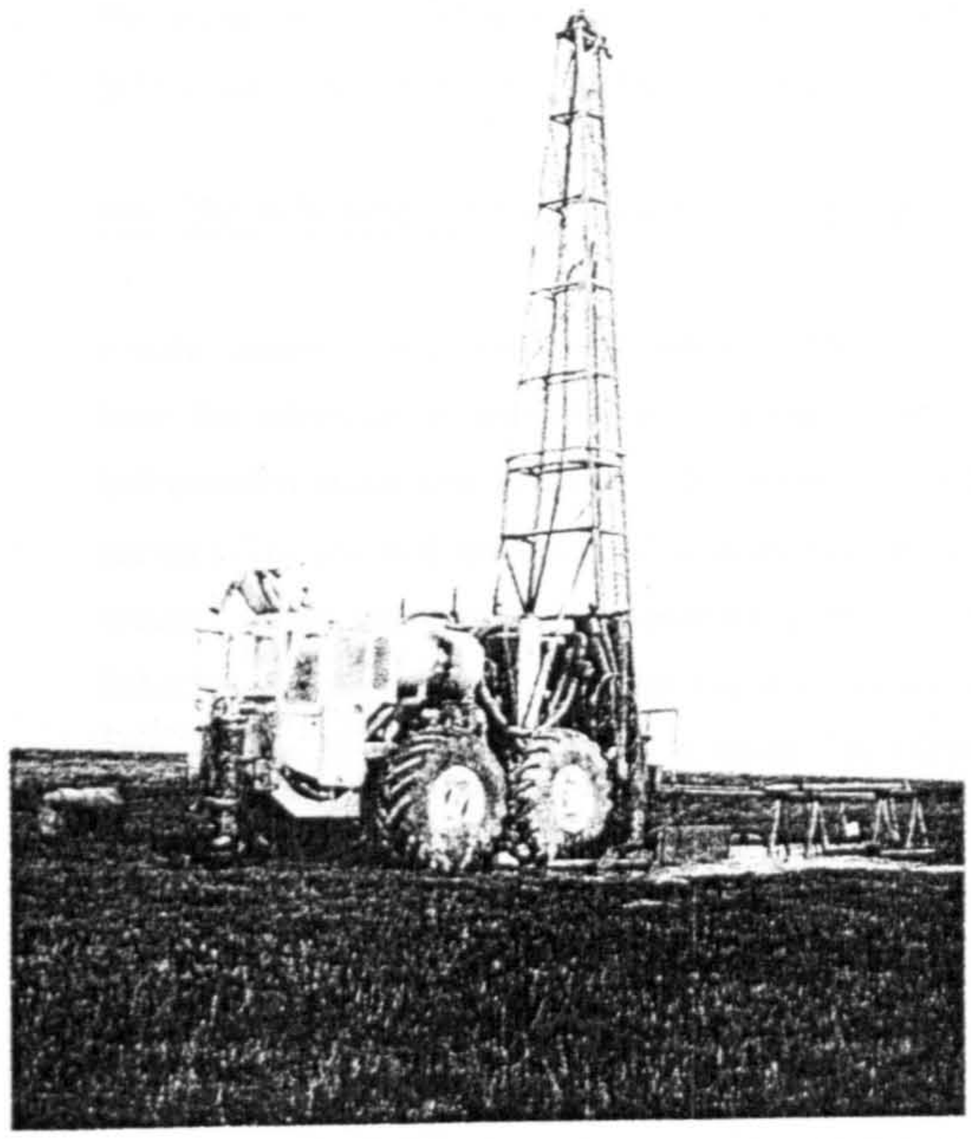

Figure 1. Rotary drilling rig prospecting for coal reserves.

Boreholes are typically arranged in a triangular grid of 50-60 metres though a closer spacing of 25-30 metres may be implemented where it is required accurately to locate coal seam sub-crops beneath overlying strata or superficial deposits; or to locate faults or washouts; or to provide some kind of statistical assessment of the extent of coal pillars in a seam which has previously been worked underground. Borehole grids may have to be distorted or remain incomplete due to the presence of buildings, roads and services, or where land is difficult to access (e.g. steep wooded valleys). Steeply dipping strata may dictate the adoption of a modified borehole grid.

Under good conditions a borehole of about 150m can be drilled in one ten hour shift, though the rate of progress will depend on the presence of soft or broken strata (e.g. sand or gravel, old workings) which normally require the insertion of steel casing to prevent collapse into the borehole. Drilling rates are also governed by the proportions of sandstone or other hard materials encountered as these take longer to penetrate than mudstones. Groundwater may also become problematic, particularly in deep boreholes where the compressed air may be insufficient to displace the

water and bring the chippings from the cutting face of the rockbit to the surface.

On rare occasions water flush drilling may be adopted, though by its very nature this drilling method is not as adaptable as air-flush to a rapid turnover of boreholes. Foam has been used to provide better readings from the borehole using biodegradable chemicals.

Coal samples are obtained from partially cored boreholes at a wider grid spacing and usually drilled alongside an original openhole borehole. Specific horizons are cored, usually coal seams with immediate roof and floor strata, using depths obtained from the driller's or geophysical log. Cores can provide accurate in-situ thicknesses of coal together with information concerning the adhesion of the roof and floor strata and the presence of bands. The material obtained is also sampled for analysis in the laboratory.

Coring is undertaken using double tube swivel type core barrels with face ejection tungsten, diamond, combination, or synthetic core bits, (Figure 2). The diameter of the core is usually "HF" sized (76mm) though "PF" sized (92mm) or "SF" sized (113mm) cores may be used where the strata is soft or disturbed.

Figure 2. Rock cores being extruded from core-barrel.

Continuous coring is adopted on some boreholes to provide information about the inter-seam strata and the cores may be logged in detail, recording natural joints and fractures, and tested for rock strength.

Down-The-Hole Geophysical (Slim-line) Logging

A more recent development in opencast prospecting has been the adoption of geophysical logging to provide information about the strata. The sonde or tool most universally adopted consists of a combination of natural gamma and two density measuring devices. Relative densities of the strata are determined by irradiation of the strata using a small low strength radioactive source of caesium-137 or iridium-192. The scattered radiation is received by detectors situated some distance from this source and shielded from it by lead.

The sonde (typically 48mm in diameter and 2.5m long) is lowered down protective casing inserted in the borehole to facilitate a smooth passage through broken strata and to prevent its loss should the winch cable snap or the sonde become trapped. The information received by the detectors is returned to the operating vehicle or unit by way of the winch cable, (Figure 3). Data is stored on cassette for later processing or is reproduced in the form of a log using a chart recorder. Coal seams and other lithologies can be identified (Figure 4) upon interpretation, and the depths and thicknesses used in the production of geological plans and sections or used as a basis for subsequent coring operations in an adjacent borehole.

Figure 3. Slim-line (geophysical) logging down cased borehole.

High

RELATIVE DENSITIES

Low

COUNTS PER SECOND

DEPTH IN METRES

56

58

60

Long Spacing Density Log

High resolution Density Log

Natural gamma log

Interpreted Coal

FIGURE 4

Part of TRISONDE GEOLOG

Other sondes are occasionally used (e.g. neutron-neutron) to provide more information about the inter-seam strata.

GEOTECHNICAL INVESTIGATION METHODS

At the feasibility investigation stage it is necessary to assess all the practicable methods of working so that the firms who will later tender for the working of the site can have as wide a choice as possible as to how to programme and design their operations. The stability of excavations and spoil mounds is considered in relation to these various methods of working. The possible effects on safety and stability of all adjacent structures and services are investigated. Earthworks and foundation conditions for any coal loading or storage structures, or highway works, or other ancillary structures are all investigated.

As stated earlier, geotechnical investigations are carried out as an extension to and continuation from prospecting investigations. Methods of exploration, sampling and testing are generally in accordance with BS 5930[1].

Rotary Drilling

Openhole drilling is carried out extensively outside proposed coaling areas to prove thicknesses of superficial deposits and the presence of shallow old workings in areas which may be used for construction of spoil storage mounds or siltation lagoons. Also the number of boreholes is increased at locations where faulting and other geological complications are proved or suspected, especially if these locations are adjacent to proposed site boundaries.

Engineering strata cores (boreholes core-sampled for their full depth from rockhead) are taken in most proposed coaling (excavation) areas as an aid to both stability and diggability assessment. Computerised logs and storage of information is a current development.

Cable Percussion Boring

Cable percussion boring is used to investigate superficial deposits particularly in relation to the siting and construction of spoil mounds and lagoons, stability of excavated slopes, and foundations for civil and structural engineering works which often arise due to the operation or restoration of opencast coal sites, (Figure 5).

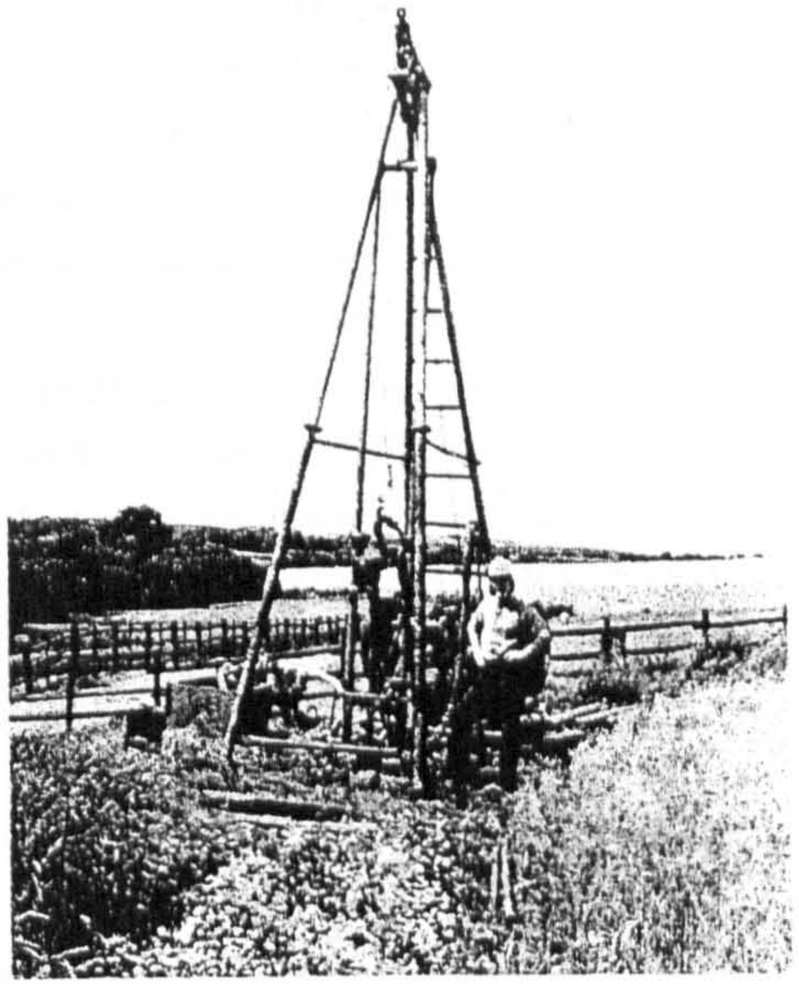

Figure 5. Cable percussion boring rig on site being investigated for future opencast coal mining.

The winch capacities of rigs are usually between 1 and 2 tonnes. The borehole is advanced by the percussive action of the clay-cutter in cohesive soils or the shell in granular soils. The occurrence of cobbles and boulders in glacial materials requires the frequent use of heavy chisels. The maximum depth of penetration attainable depends upon the nature of the ground being investigated and to some degree on the skill and experience of the operators. Using several reducing sizes of casing (e.g. 250mm, 200mm and 150mm diameter) boreholes in excess of 60 m depth are claimed to be possible. However, where glacial deposits are the materials being penetrated, around 40 m depth seems to be the practical limit.

Sampling programmes are determined in relation to the types of soils encountered and the purpose of the investigation. Where the exploratory boring is through heavily overconsolidated gravelly clays of glacial origin, the use of standard U100 open-drive

tubes is the only practical method of sampling. Continuous sampling is often carried out in relation to the stability investigations for excavated slopes and spoil mounds. Disturbed samples and water samples are also obtained, and Standard Penetration Testing is carried out in granular soils and weathered bedrock. Very occasionally soft alluvial soils are encountered and may require the use of piston-sampling and in-situ shear vane testing.

Triaxial compression shear strength testing is carried out extensively, to determine both total stress and effective stress shear strength parameters. Some shear-box testing is also carried out for residual shear strength purposes. Other soil mechanics laboratory work includes consolidation testing, compaction testing, and a whole range of classification and chemical testing.

In association with cable percussion boring shallow pits are often excavated using hand tools to locate underground services. Also pits are sometimes excavated by backhoe to investigate foundation conditions for new carriageways or shallow footings etc.

Borehole Instrumentation and In-situ Testing

Instrumentation is installed in boreholes either to monitor groundwater levels or to monitor ground movements.

Groundwater investigations mainly involve the installation of simple "Casagrande" type piezometers to allow the recording of water levels over a long period of time. The piezometer porous tip is surrounded by filter media with bentonite seals above and below, and is positioned so that the recorded water level applies to a particular stratum or aquifer. Standpipes (similar to piezometers but with the filter media spanning the full depth of the borehole) are often installed in shallow borings, particularly in superficial deposits or in old opencast backfills. In-situ permeability measurements are obtained either through permeability testing in piezometers or packer tests, or very occasionally by pumping from deep wells.

Ground movements can result from a number of causes including excavation slope instability,heave adjacent to spoil mounds, collapse of old underground workings, and settlement in areas of old opencast backfill. Wherever any of these conditions exist and ground movements could have serious consequences, then borehole instruments designed to monitor these movements may be installed. For slope stability monitoring simple mechanical slip indicators (sliprods)or inclinometers are often chosen as these can be used to locate planes of differential (shear) movements. Tensioned-wire extensometers have been used for several applications including slope stability monitoring, underground workings collapse and settlement of backfill. Magentic extensometer settlement gauges have been found particularly useful for monitoring backfill settlements. Papers by Hughes and McLean [2], Singh et al [3], and Charles et al [4] describe the construction and installation of several types of borehole instruments used in opencast mining applications.

INFORMATION FROM SUB-SURFACE INVESTIGATIONS

The results of all the prospecting investigations are assessed and are presented in the form of a Final Site Report which includes geological plans and sections.

The stability aspects of working any particular site are assessed from a comprehensive study of all the ground investigation information. The Geotechnical Stability Report is prepared in which the stability of excavation faces, groundwater conditions and factors relating to the safety of adjacent structures and services are all discussed. In addition a two part Code of Practice document [5] is prepared which relates to the siting and construction of spoil mounds and is designed to ensure that the technical standards required by the Mines and Quarries (Tips) Act (1969)[6] and Regulations (1971) [6] are applied to opencast coal sites. A more comprehensive account of stability and other geotechnical matters relating to the operation of opencast coal sites is given by Hughes and Leigh [7].

ACKNOWLEDGEMENTS

The Authors wish to thank British Coal, Opencast Executive for permission to present this paper, and to thank several colleagues who have given much helpful criticism and advice during the preparation. The views expressed are those of the Authors and do not necessarily reflect the views or policy of British Coal.

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PAPER C10 NORTHERN BACKFILL 2

Reed, S.M., Hughes, D.B. and Singh, R.N. 1987.

Backfill settlement of restored strip mine site – case histories.

International Journal of Mining and Geological Engineering, Vol.5, No.2, 161-169.

International Journal of Mining and Geological Engineering, 1987, **5**, 161–169

SHORT COMMUNICATION

Backfill settlement of restored strip mine sites – case histories

Introduction

The paper presents an analysis of backfill settlement observations conducted upon a number of restored strip–mine sites in the North-East of England. The work is aimed at developing a better understanding of the stability of restored mine backfills from the point of view of structural development. The project is specifically concerned with the effect on fill movement of the re–establishment of the groundwater regime after the completion of surface mining and in particular the occurrence of collapse settlements associated with groundwater recovery within the backfill mass. Groundwater recovery commences on the termination of pumping in the area of the final void of a surface mine, although drawdown effects may have enabled the water to rise to some degree through fill which is distant from the final void. The rate of recharge is dependent on a number of factors, such as local hydrology, nature and permeability of the spoil and climatic conditions, (Singh *et al.*, 1985a; Singh *et al.*, 1985b).

The observation of backfill stability in the North-East of England started in the 1960s when Kilkenny (1968) investigated the suitability of restored surface coal mine sites for the purposes of structural development. He investigated relatively shallow backfill areas, and concluded that the settlement rates followed a semi-logarithmic decay with respect to time. A detailed investigation on an opencast site at Horsley, Northumberland, (Charles *et al.*, 1977, 1983), monitored the effects of water recovery within the backfill on settlement rates, indicating principally the inducement of collapse settlements in backfill materials. The project also reported heaving movements associated with removal of fill surcharges and reduced settlement associated with presaturation of fill in the proximity of settling lagoons.

Site descriptions and results

The monitoring scheme on Site A commenced in early 1982. The site, a dragline operation backfilled in mid-1974, had a sandstone–mudstone type fill, the geological sequence worked being largely arenaceous. Two million tonnes of coal were extracted from the site at an overburden to coal ratio of about 20:1.

The project consisted of the monitoring of the settlement of a sewerage pipeline constructed

Keywords: Backfill settlement; strip mining; groundwater levels.

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over the mine backfill. Instrumentation consisted of the installation of three magnetic extensometers/piezometers in the fill complemented by three sets of 10 surface levelling stations. Manhole covers placed above the sewerage line also acted as surface levelling stations. The instrumentation layout is illustrated in Fig. 1.

The design of the pipeline was such that some degree of settlement could be tolerated by the introduction of flexible joints between individual pipe sections.

The time elapse between the date of backfilling, (mid 1974) and the commencement of monitoring, (early 1982) of 7.5 years was to prove extremely important in the analysis of the results to the present date. Movements have been recorded of up to a maximum of 15 mm; stations having shown either small settlement or heaving movements. The lack of water within holes E1 and E2 is in sharp contrast to the levels recorded on instrument E3. Fig. 2 illustrates the groundwater levels and settlement characteristics for instrument E3 against time.

Whilst trends would appear to exist between settlement and water levels from inspection of Fig. 2, it is considered dangerous and probably meaningless to 'over–analyse' these results. Movements of fill in the region of ± 12 mm in about 16 m of fill depth show for the purposes of this exercise that the pipeline is not being subjected to adverse differential backfill movements. Little correlation can be drawn between recovery and settlement and this is probably due to the fact that monitoring commenced 7.5 years after backfilling had been completed. From the E3 instrument results, however, it can be stated that after an uncertain time period a fluctuating water table will cease to cause further significant collapse settlements.

Site B was worked between 1957 and 1973 producing about 7 million tonnes of coal from a dragline operation. The backfill is a typical sandstone/mudstone fill common in the area. On the completion of areas of backfilling, three surface levelling traverses were installed. Fig. 3 shows the position of the main traverse together with approximate depths of fill, and piezometer location (A).

The water levels at the piezometer are illustrated in Fig. 4a. The figure shows that the water levels over this part of the site were recovering slowly from the start of monitoring up until May/June 1982. Fig. 4b shows the settlement versus time curves for all the stations over the traverse. It can be seen that settlement rates retarded around the time of the completion of groundwater recovery thus indicating that settlement and groundwater were related. Following recovery, the groundwater levels can be seen to be dropping and this corresponds to a period of

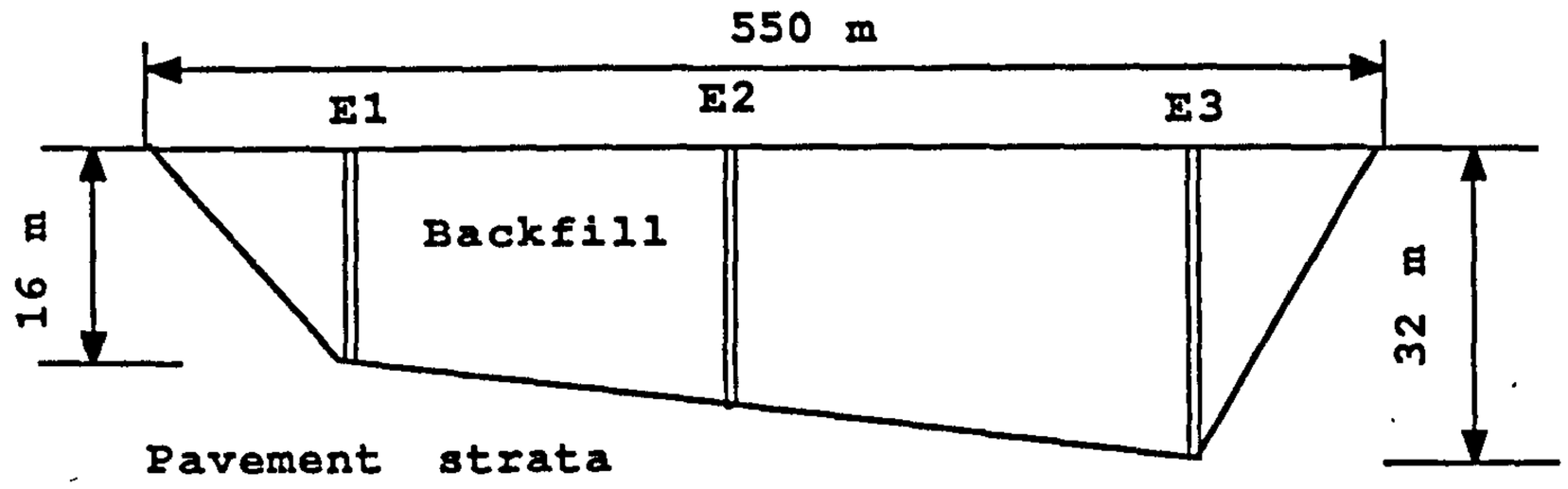


Fig. 1. Instrumentation layout at dragline site A.

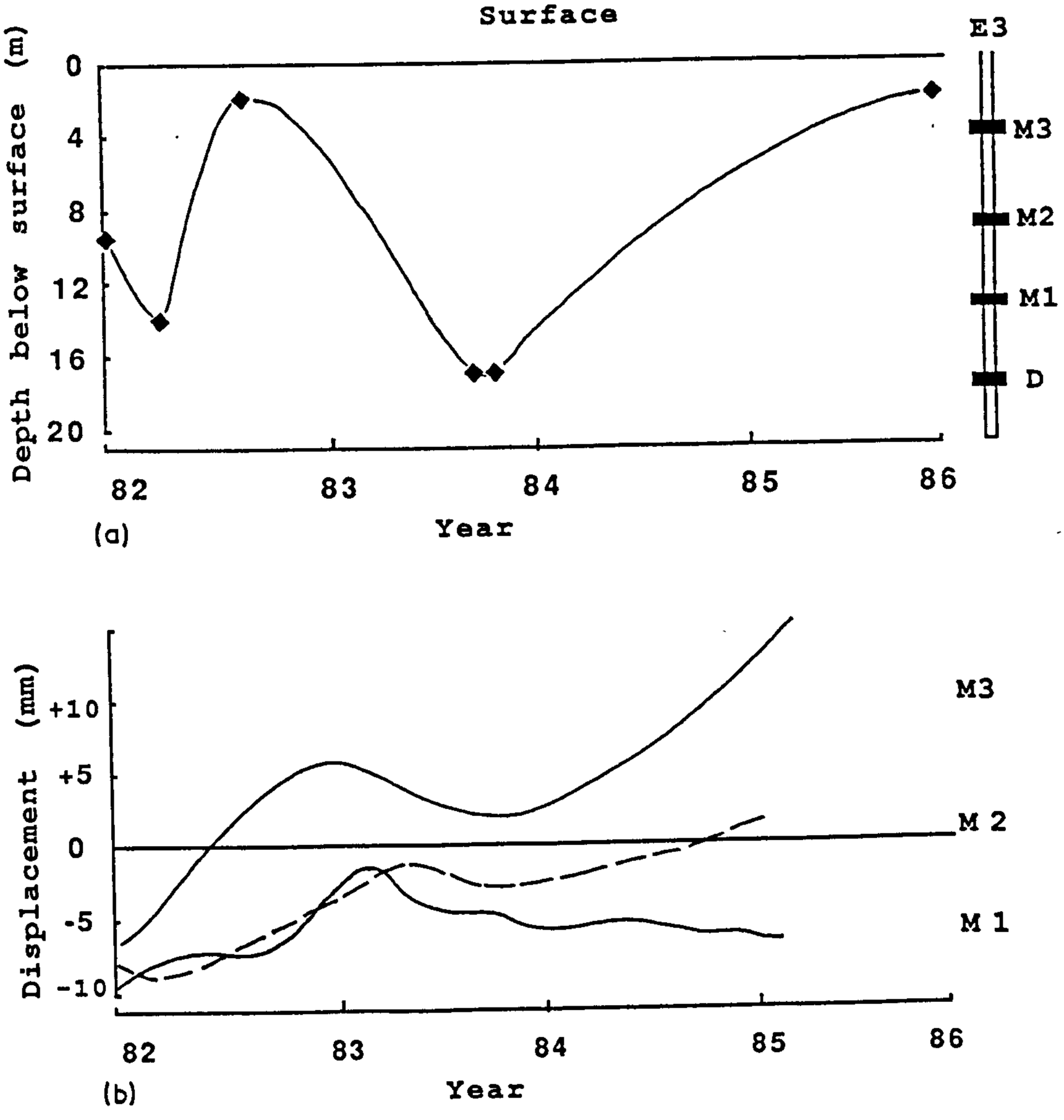


Fig. 2. (a) Water level records, extensometer E3; and (b) settlement records, extensometer E3.

negligible backfill movement. After this period, groundwater levels again rise and settlement is seen to restart. It must be noted that the levels in the piezometer are higher than the restored level of the traverse, (by up to 10 m), some 320 m distant. It is, however, considered that it is the trends observed that are important and that similar water level changes have occurred within the fill material.

Total settlement results for this traverse are much as would be expected, i.e. maximum settlements have been recorded on stations 1, 2 and 3, the stations in the newest fill material. The

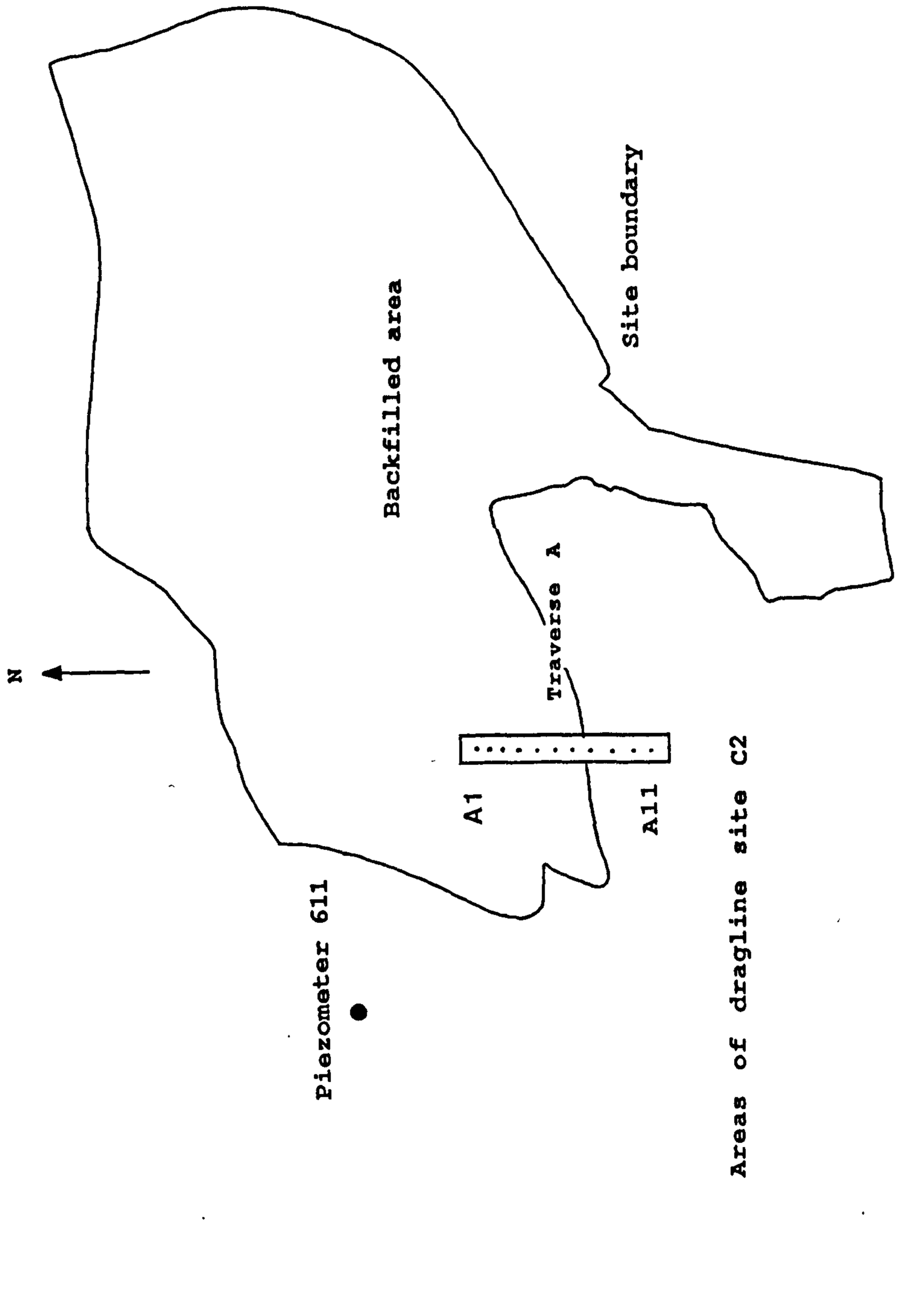
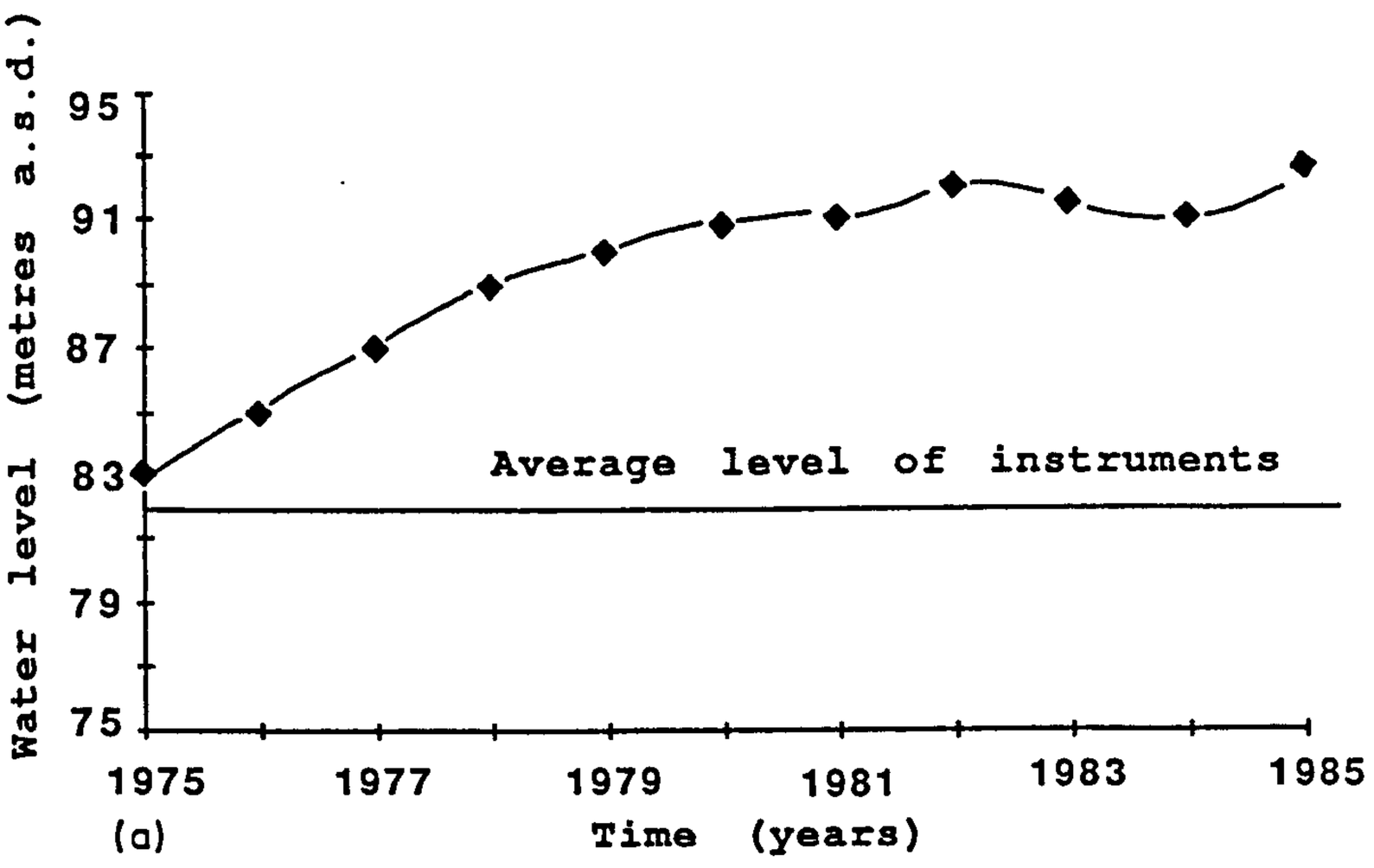


Fig. 3. Instrumentation layout, dragline site B.



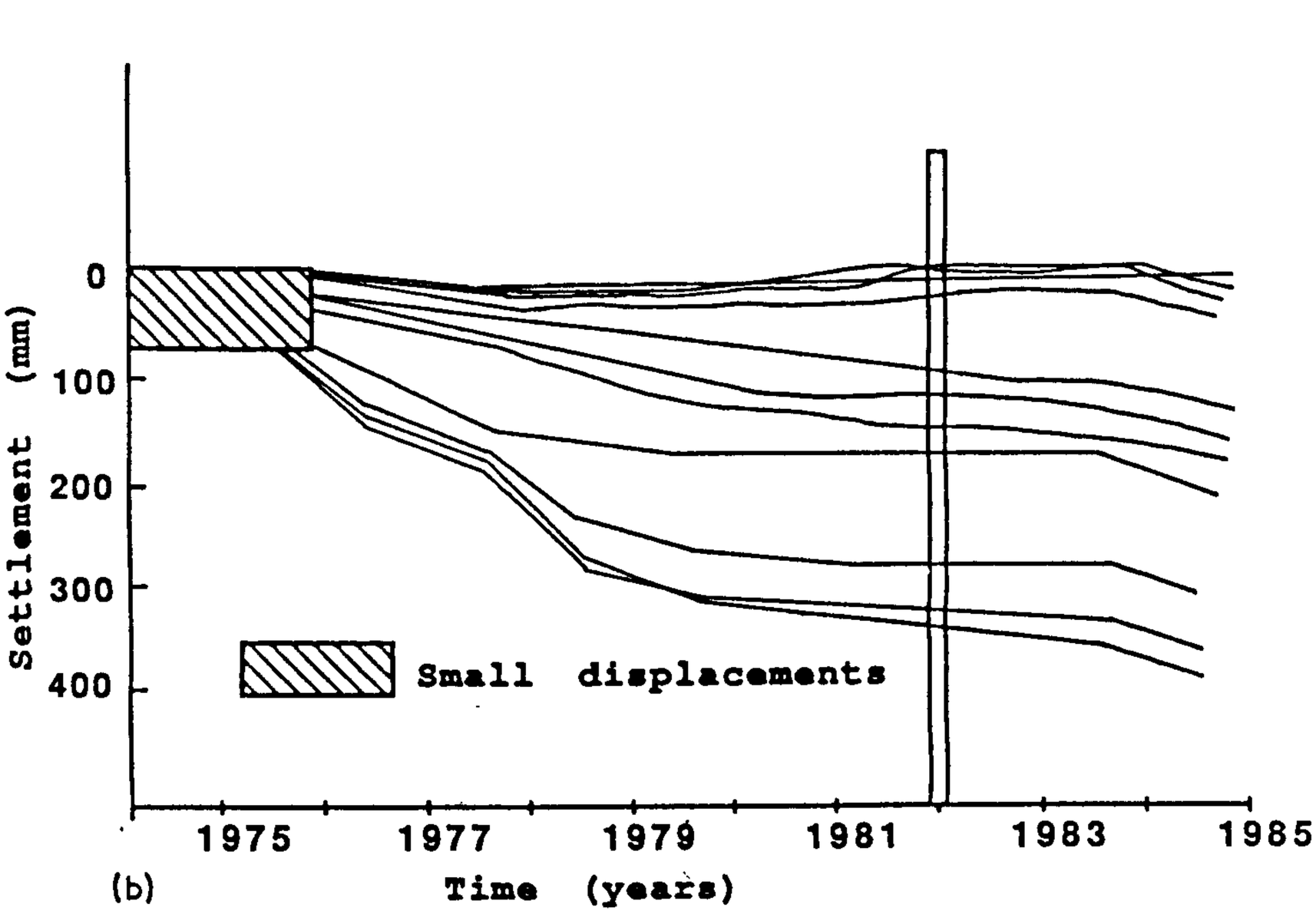


Fig. 4. (a) Water levels in local piezometers 611, dragline site B; and (b) settlement records, traverse A, dragline site B.

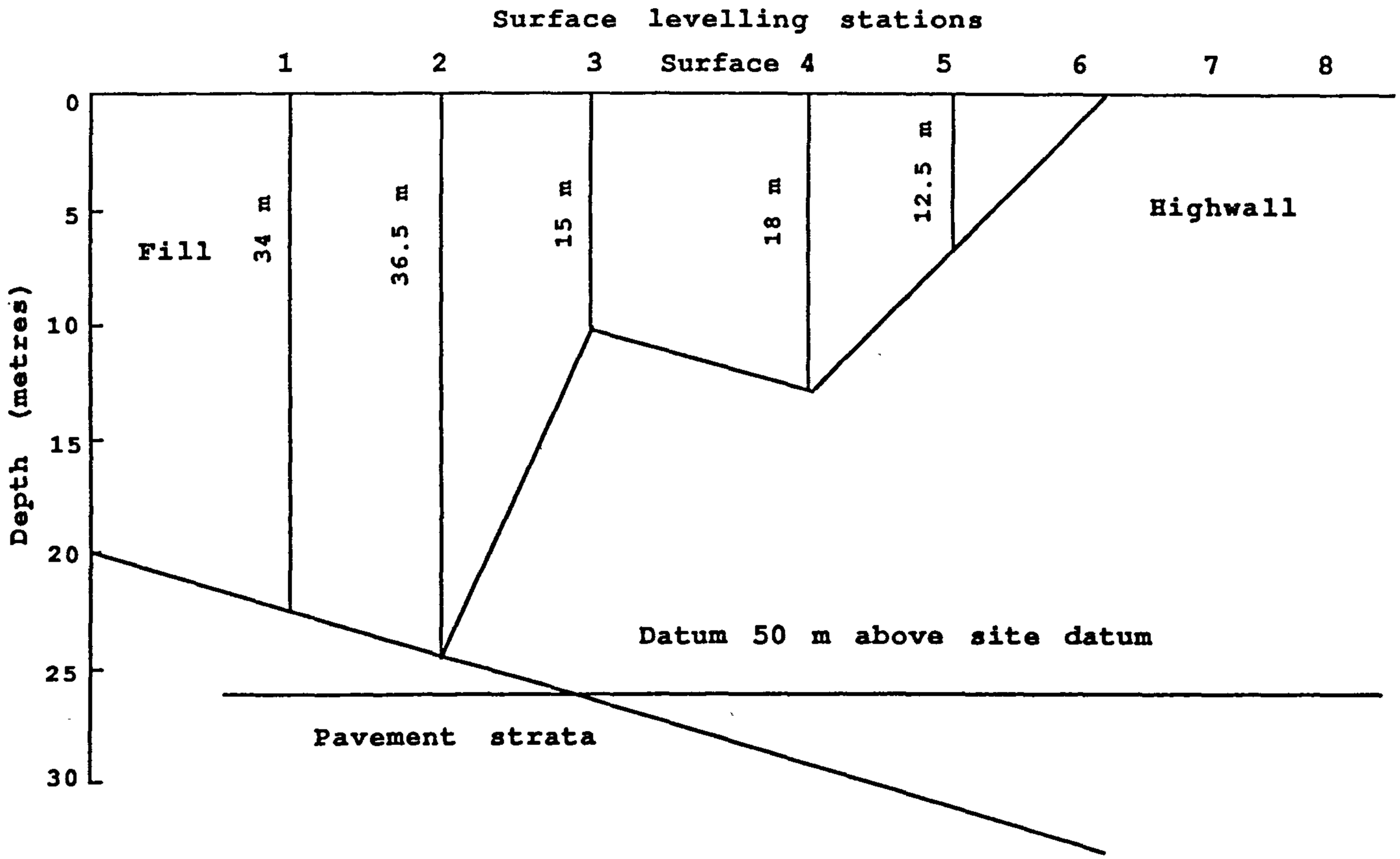


Fig. 5. Instrumentation layout, dragline site C.

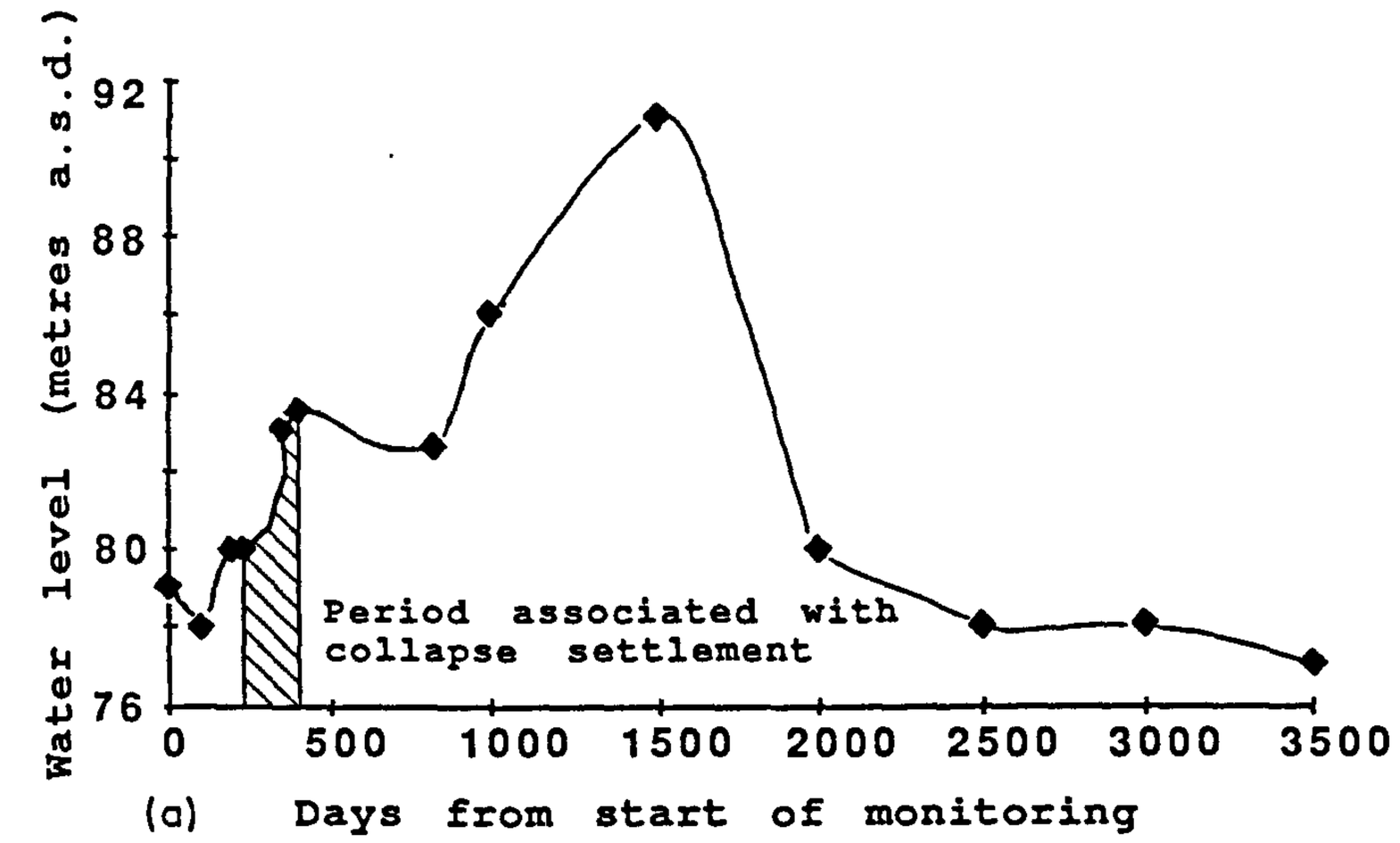


Fig. 6. (a) Water levels in local piezometers 2002, dragline site C; and (b) settlement results, dragline site C.

least settlement has been recorded at station 11 which along with stations 5 and 6 have undergone negligible movement. Station 11 is thought to be on the oldest fill in the traverse as well as being the shallowest, (~6 m).

Site C produced 2.5 million tonnes of coal by dragline methods over the period of 1971 to 1979. The nature of the backfill was a typical mudstone–sandstone fill with an overburden to coal ratio of about 17:1. Instrumentation in the form of 8 surface levelling stations commenced in late 1975 over an area of the site. Water levels in this area were dominated by pumping in a local colliery shaft. The instrumentation scheme is illustrated in Fig. 5. Groundwater patterns have been inferred from the water levels in a local piezometer 2000 metres away (Fig. 6a).

The settlement results for the instrumentation scheme are presented in Fig. 6b. The maximum historical settlement recorded has been 451 mm in 18 m of fill, (2.51%), whilst the maximum percentage settlement has been 2.72%. (340 mm in 12.5 m of fill).

The total settlement results for this site have produced some interesting results summarized as follows,

(i) The minimum fill settlement of 288 mm has been recorded in the maximum fill depth of 36.5 m, (0.79%).

(ii) In the highwall area the degrees of settlement appear unrelated to fill thickness.

The most likely explanation for these phenomena is probably that the standard of initial compaction of the fill in close proximity to highwalls has a greater variance than say for the rest of the mine. The physical presence of the solid wall must also prevent lateral movements within the fill which can reduce in turn vertical displacements.

An abnormal phase of settlement was found to have occurred on all stations over the fill during the period of 270–361 days from the start of monitoring. Settlement rates on either side of this period appear to be uniform and have continued in this uniform fashion to the present day. Of particular note is the fact that settlements on all stations over the fill are continuing at the rate of 10 mm per year, 9.5 years after monitoring commenced.

The occurrence of a sudden and dramatic increase in the rate of settlement on all stations sited over fill would appear to be related to a phase of groundwater recovery. The fill in this area is shallow and it could be reasonable to suggest that groundwater recovery could have been completed in the period of 91 days after day 270.

The piezometer results show that in the period in question, the water levels rose by about 4 m within this hole to attain a level of 16.45 m below Ordnance Datum. The shaft water levels also show a rise in water levels which occurred over this period. These results would appear to leave no doubt that groundwater recovery occurred in the Dragline Site C fill to cause the increased settlement rates.

Following the large settlements of this period, a reduced settlement rate has continued in the fill for 8.5 years. This implies that even following the completion of groundwater recovery, significant movements can occur in shallow fills for a considerable time. It is unfortunate that this particular site was not instrumentated with magnetic extensometers and piezometers to observe whether subsequent movements have occurred in saturated or unsaturated fill materials.

Acknowledgements

The authors would like to thank Mr J. Tomlinson and Mr A. Mclean of British Coal, Opencast Executive for their co-operation and continual support for this project. Gratitude is extended to Professor T. Atkinson, Head of Department of Mining Engineering, Nottingham University for his support. Opinions expressed are those of the Authors and do not necessarily reflect the opinions of British Coal.

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PAPER C11 NORTHERN BACKFILL 3

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Research into the stability of restored opencast coal mine sites in the North East of England.

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International Journal of Surface Mining 1 (1987): 257-266

Research into the stability of restored opencast coal mine sites in the north east of England

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ABSTRACT

The paper presents an analysis of backfill settlement observations conducted at a number of opencast mine sites in the North-East of England. The work is aimed at developing a better understanding of the stability of the restored mine backfills from the point of view of structural development. The project is specifically concerned with the effect on fill movements of the re-establishment of groundwater regime on the completion of surface mining, in particular the occurrence of collapse settlements associated with groundwater recovery within the backfill mass.

INTRODUCTION

In recent years opencast mine sites are being increasingly restored for urban development. Therefore, it is important to understand the mode and mechanism of instability in the backfill mass, as a consequence of structural development so as to avoid any post-development stability problems. A number of factors affect instability of the backfill including some of the following points:-

- Nature of backfill materials
- Method of placement
- Size, shape and depth of the excavation
- Degree of compaction
- Groundwater recovery in the backfill mass
- Time

Groundwater recovery commences on the termination of pumping in the area of the final void of a surface mine, although drawdown effects may have enabled the water to rise to some degree through fill which is distant to the final void. The rate of recharge is dependent on a number of factors such as local hydrology, nature and permeability of the spoil, climatic conditions, (Singh, Denby and Reed, 1985).

A summary of the sites included in this survey is presented in Table 1, along with details of instrumentation. The initial results from this work were presented by Singh, Reed, Denby and Hughes (1985).

PREVIOUS OBSERVATIONS IN THE NORTH-EAST REGION

The observations of backfill stability started in the 1960's when the suitability of restored surface coal mine sites for the purposes of structural development was investigated, (Kilkenny, 1968). The project investigated relatively shallow backfill areas, and concluded that the settlement rates of backfill materials may follow a semi-logarithmic decay with respect to time. A detailed investigation on an opencast site at Horsley, Northumberland, (Charles et al, 1977, 1983) monitored the effects of water recovery within the backfill on settlement rates, indicating principally the inducement of collapse settlements in backfill materials. The project also reported heaving movements associated with removal of fill surcharges and reduced settlement associated with presaturation of fill in the proximity to settling lagoons. The subsequent sections of this paper report the findings from groundwater and backfill settlement surveys conducted upon 6 other opencast coal mine sites in the region.

DRAGLINE SITE A, INSTALLATION OF SEWAGE PIPELINE

Instrumentation:- The monitoring scheme on Site A commenced in early 1982. The site, a dragline operation backfilled in mid-1974 had a sandstone-mudstone type fill, the geological sequence worked being largely arenaceous. Two million tonnes of coal were extracted from the site at an overburden to coal ratio of about 20:1.

The project consisted of the monitoring of the settlement of a sewage pipeline constructed over the mine backfill. Initially it was considered that the sewage line be diverted around the highwall of the pit thus avoiding passing over fill material. However, this scheme was considered very expensive in terms of material and manpower costs. Consequently British Coal agreed to divert the line directly over the fill and introduce a settlement monitoring programme to ensure that abnormal ground movements could be detected and not adversely affect the condition and performance of the pipeline.

The instrumentation consisted of the installation of three magnetic extensometers/piezometers in the fill complemented by three sets of 10 surface levelling stations. Manhole covers emplaced above the pipeline also act as surface levelling stations. The instrumentation layout is illustrated in Figure 1. The design of the pipeline was such that some degree of settlement could be tolerated by the introduction of flexible joints between individual pipe sections.

Surface Displacements:- The time elapse between the date of backfilling, (mid 1974) and the commencement of monitoring (early 1982) of 7.5 years was to prove extremely important in the analysis of the results to the present date. Movements have been recorded of up to a maximum of 15 mm, stations have in fact shown either small settlement or heaving movements.

Groundwater and Settlement:- The lack of water within holes E1 and E2 is in sharp contrast to the levels recorded on instrument E3. Figure 2 illustrates the groundwater levels and settlement characteristics for instrument E3 against time.

Whilst trends would appear to exist between magnets and water levels from inspection of Figure 2, it is considered dangerous and probably meaningless to "over-analyse" these results further. Movements

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Table 1. Summary of Site Investigations

Site Description	Instrumentation Details
Dragline Site A. 15 - 30 metres depth of fill. Sandstone-Mudstone.	23 Surface levelling stations and 3 magnetic extensometers/ piezometers monitoring the line of a sewage pipe constructed over the fill.
Dragline Site B. Sandstone-Mudstone fill. Depths 15 - 30 m.	3 traverses totalling 30 surface levelling stations over fill area. Correlated to local piezometers for groundwater levels.
Dragline Site C. Sandstone-Mudstone fill. Depths in area of instrumentation up to 35 metres.	8 surface levelling stations installed over highwall area. Correlated to local piezometers for groundwater levels.
Shallow Truck-Shovel Site D. Sandstone-Mudstone fill. No water.	9 surface levelling stations monitoring vertical and lateral fill movements.
Dragline Site E. Sandstone-Mudstone fill. Damage to concrete lined channel and lake constructed on restored fill surface.	None
Truck-Shovel Site F. On side of hill. Shallow depth 11 m, Sandstone-Mudstone fill.	None

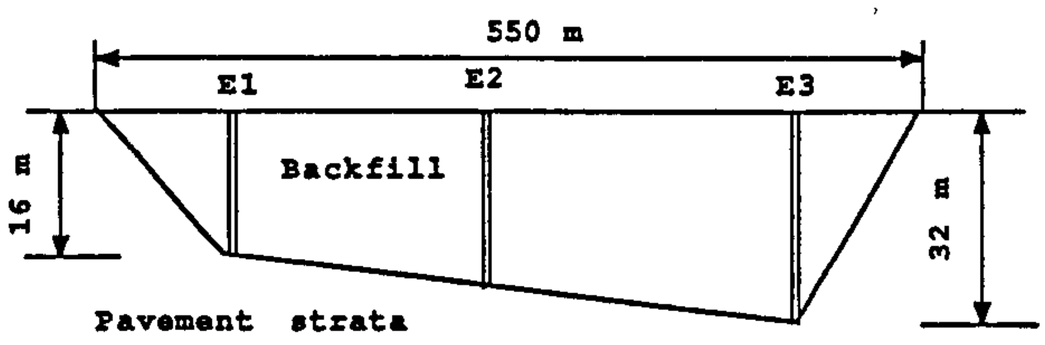

Figure 1. Instrumentation layout; Dragline Site A.

of fill in the region of 12 mm in about 16 m of fill depth show for the purposes of this exercise that the pipeline is not being subjected to adverse differential backfill movements. No correlation can be drawn between recovery and settlement and this is in no doubt due to the fact that monitoring commenced 7.5 years after backfilling had been completed. From the E3 instrument results however it can be stated that after an uncertain time period a fluctuating water table will cease to cause further significant collapse settlements.

DRAGLINE SITE B

Site B was worked between 1957 and 1973 producing about 7 million tonnes of coal from a dragline operation. The backfill is a typical sandstone/ mudstone fill common in the area. On the completion of areas of backfilling, three surface levelling traverses were installed. Figure 3 shows the positions of the traverses over the fill.

Traverse A passes over both Site B and an adjacent opencast mine site, (referred to hereafter as Site B2), which had completed coaling in 1956 just prior to the commencement of Dragline Site B. The traverse is illustrated in Figure 3 along with the position of a piezometer from which an approximate idea of levels of water within the fill may be obtained. The water levels in this instrument are displayed graphically in Figure 4, showing that the levels over this part of the site were recovering slowly from the start of monitoring up until May/June 1982. Figure 5 shows the settlement versus time

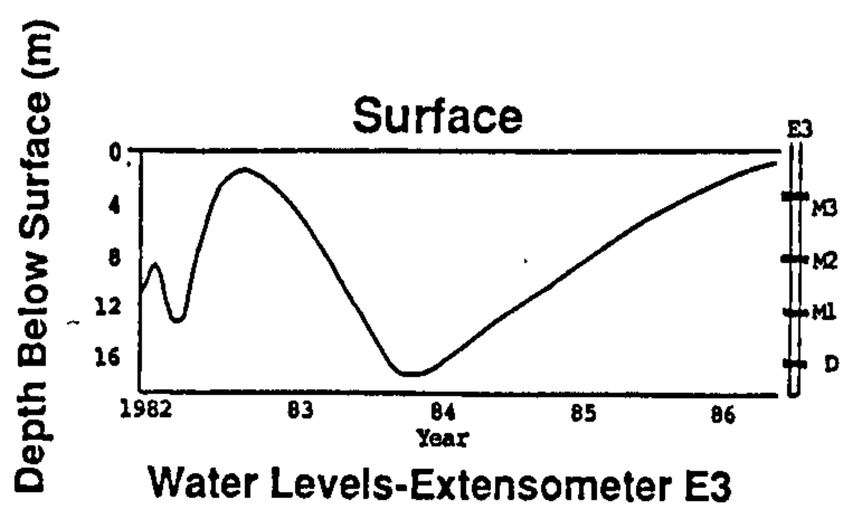

Figure 2. Summary of Results; Extensometer E3, Dragline Site A.

curves for all the stations over the traverse. Total settlements are detailed in Table 2.

It can be seen that settlement rates retarded around the time of the completion of groundwater recovery thus indicating that settlement and groundwater were related.

Following recovery, the groundwater levels can be seen to be dropping and this corresponds to a period of negligible backfill movements. After this period, groundwater levels again rise and settlement is seen to restart. It must be noted that the levels in the

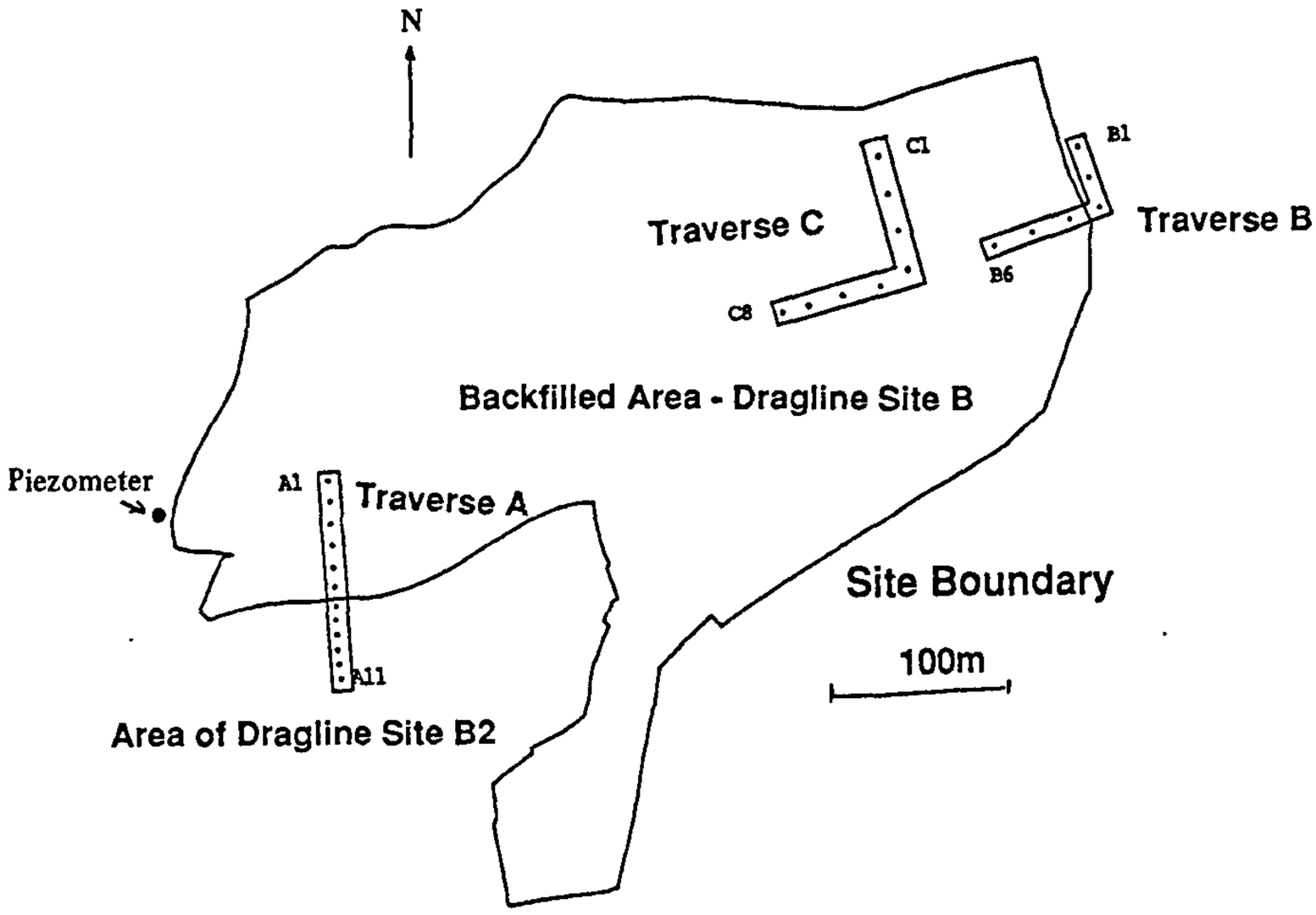

Figure 3. Instrumentation Layout; Dragline Site B.

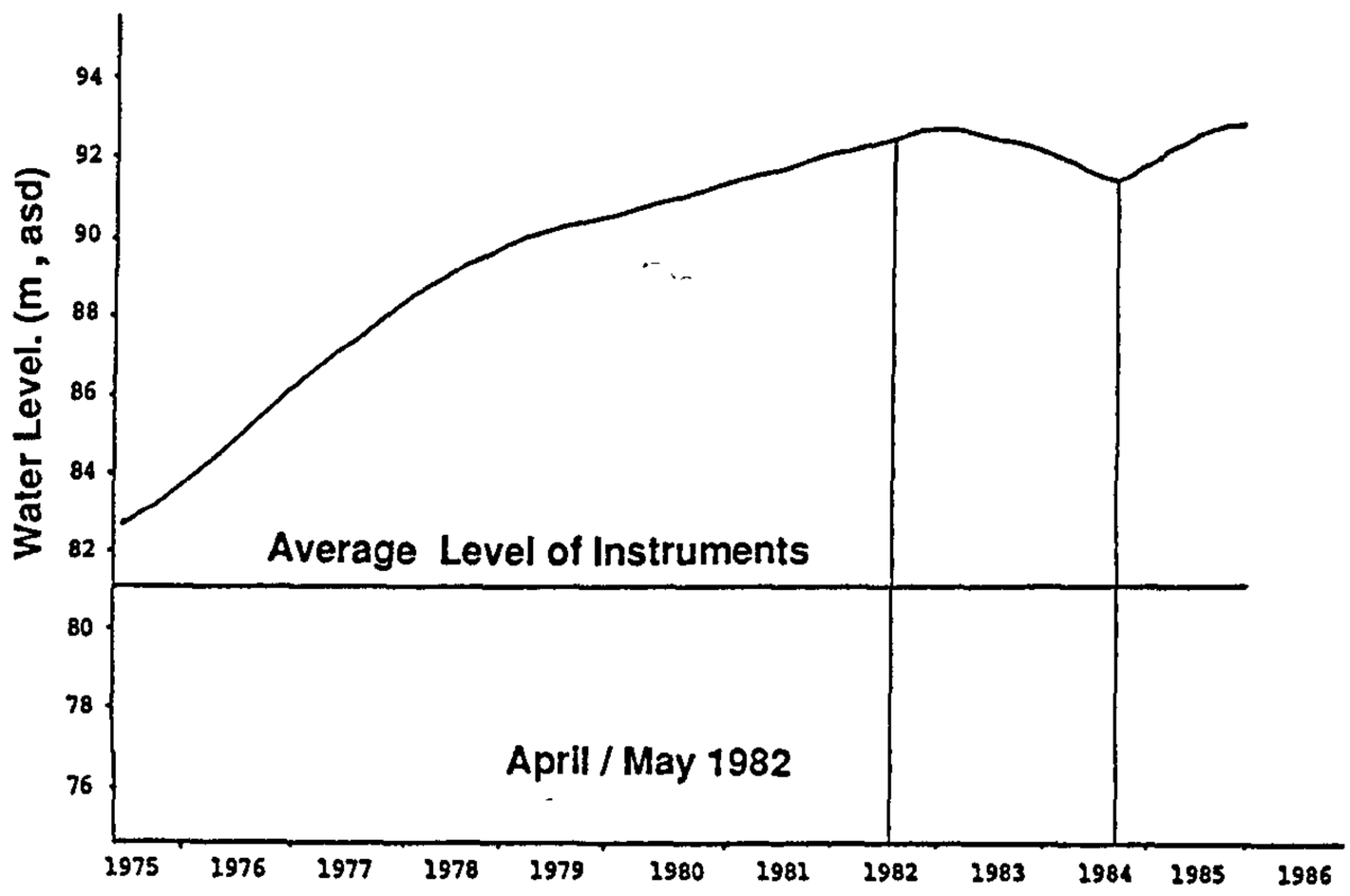

Figure 4. Water levels in local Piezometer; Dragline Site B.

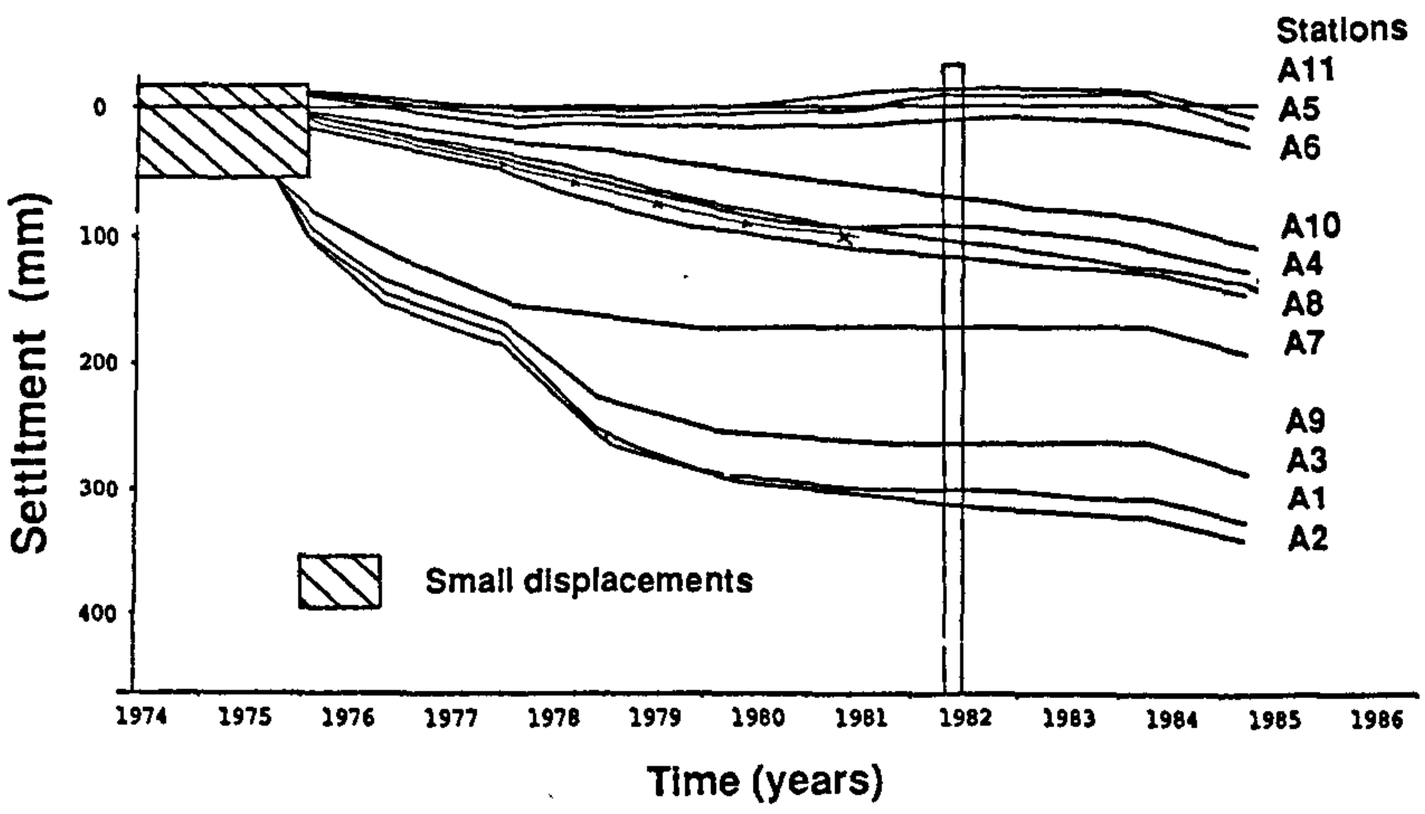

Figure 5. Settlement records, Traverse A; Dragline Site B.

Table 2. Summary of Settlement Results, Traverse A; Dragline Site B, over 11 year period

Station	Approx. Depth of Fill (m)	Total Settlement (mm)	Settlement mm/m depth	Comments
A1	18	328	18	Closest to 611
A2	21	339	16	
A3	24	290	12	
A4	21	110	5	
A5	uncertain	8	-	Border of Sites B-B2
A6	uncertain	26	-	
A7	35	148	4	
A8	18	129	7	
A9	7	195	28	
A10	7	89	12	
A11	6	+16	+3	Oldest fill

piezometer are higher than the restored level of the traverse (by up to 10 m), some 320 m distant. It is however considered that it is the trends observed that are important and that similar water level changes have occurred within the fill material.

Total settlement results for this traverse are much as would be expected, ie maximum settlements have been recorded on Stations 1, 2 and 3, the stations in the newest fill material. The least settlement has been recorded upon Station 11 which along with Stations 5 and 6 have undergone negligible movement. Station 11 is thought to be on the oldest fill in the traverse as well as being the shallowest (c. 6 m). The depths of fill under Stations 5 and 6 are thought to be shallow as well owing to this being on the border on the B and B2 sites.

Settlement results for Traverse B are presented in Figure 6. A summary of total settlements is given in Table 3. The traverse is illustrated in Figure 3 showing that 3 of the initially 6 monitored stations were positioned on solid ground. Monitoring has been performed in two stages, from 1969 until 1976 and from 1983 to the present day. As a conse-

Table 3. Summary of Settlement Results, Traverse B; Dragline Site B

Station	Approx. Depth of Fill (m)	Total Settlement (mm)	Settlement mm/m depth
B1	0	0*	-
B2	0	+5*	-
B3	0	+7*	-
B4	35	347*	10
B5	55	657*	12
B6	55	402**	7

**Readings taken over 7 years

*Readings taken over 15 years

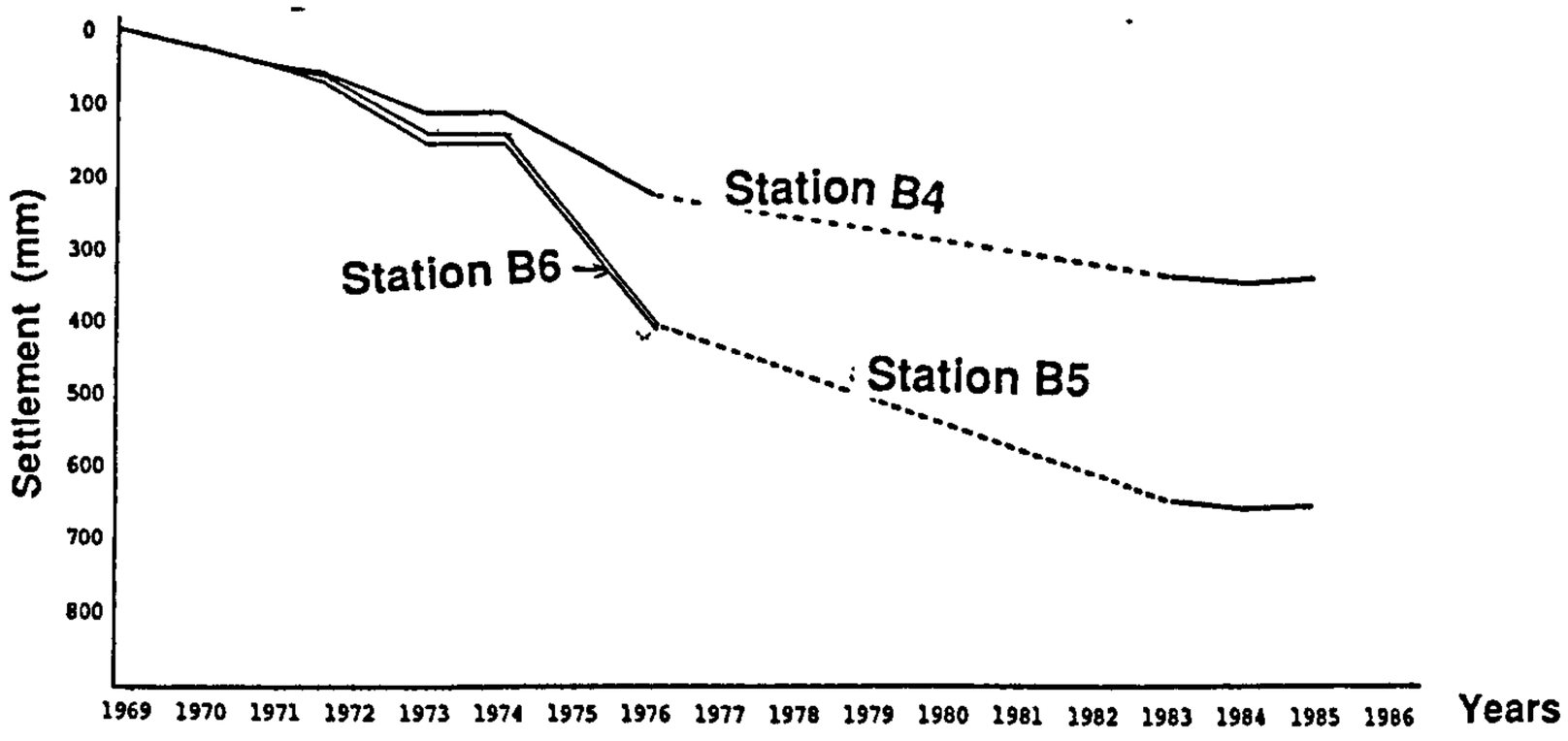

Figure 6. Settlement records, Traverse B; Dragline Site B.

Table 4. Summary of Settlement Results, Traverse C; Dragline Site B, over 2 year period

Station	Total Settlement (mm)
C1	-1
C2	+1
C3	-1
C4	-3
C5	-9
C6	-8
C7	-21
C8	-5

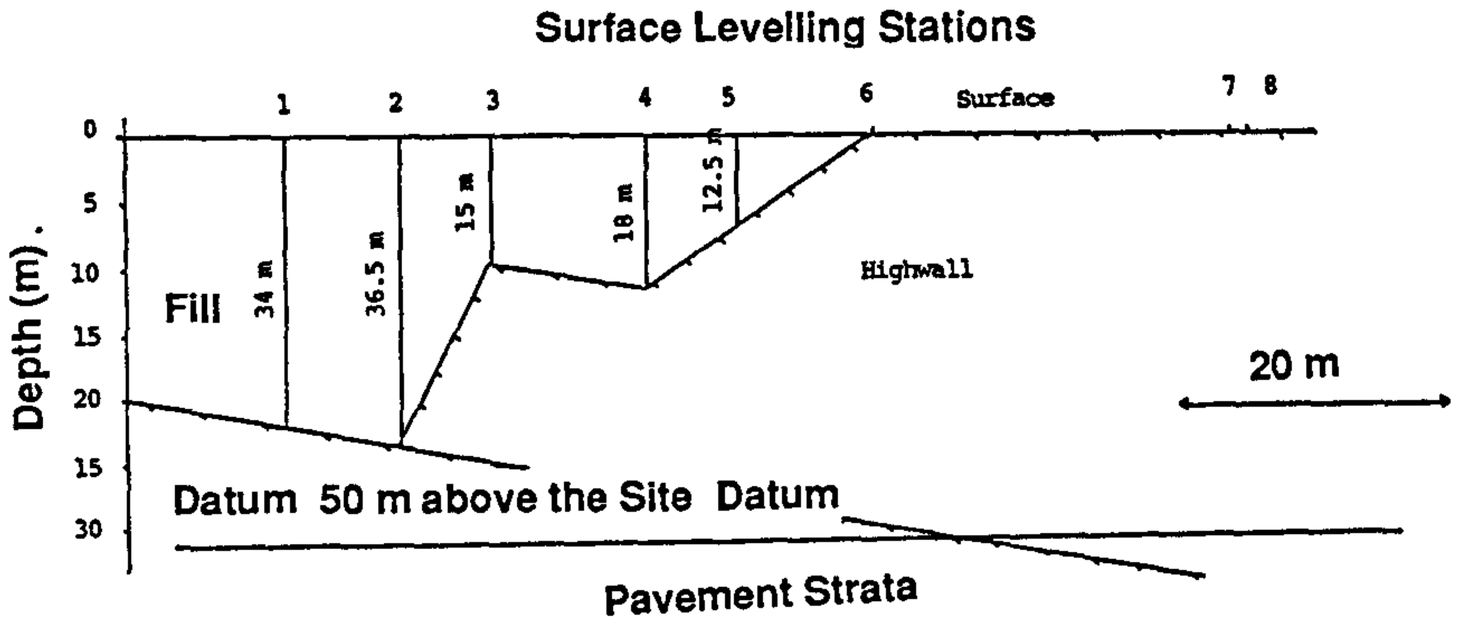

Figure 7. Instrumentation Layout; Dragline Site C.

quence of this a great deal of the settlement data has been lost. The absence of a suitable piezometer within the area together with the loss in settlement readings has meant that the results cannot be correlated to groundwater recovery. As the trend of the settlement curves when compared to Traverse A is similar, then it is considered that groundwater recovery has occurred within the fill and consequently induced settlements.

Traverse C has been monitored from 1983 until the present day and consists of 8 surface levelling stations over the fill. The total settlements are presented in Table 4 and these show a maximum settlement recorded of 21 mm on Station C7. This would appear to show that a great deal of settlement has occurred prior to monitoring.

DRAGLINE SITE C, AFFECTED BY UNDERGROUND MINE PUMPING

Site C worked 2.5 million tonnes of coal by dragline methods over the period of 1971 to 1979. The nature of the overburden being a typical mudstone-sandstone fill with an overburden to coal ratio of about 17:1. Instrumentation in the form of 8 surface levelling stations commenced in late 1975 over an area of the site. Water levels in this area are being dominated by pumping in a local colliery shaft. The instrumentation scheme is illustrated in Figure 7. Groundwater patterns have been inferred from both the water levels in the colliery shaft about 1000 m distant and in a local piezometer 2000 metres away.

Settlement Observations:- The settlement results for the instrumentation scheme are presented in Figure 8. Table 5 presents the results for all the stations expressed for the period of 9½ years of monitoring (1975-1985) in the form of percentage of fill depth settlements. The maximum settlement recorded has been 451 mm in 18 m of fill, (2.51%), whilst the maximum percentage settlement has been 2.72%, (340 mm in 12.5 m of fill).

The total settlement results for this site have produced some interesting results summarised as follows:

(i) The minimum fill settlement of 288 mm has been recorded in the maximum fill depth of 36.5 m (0.79%).

(ii) In the highwall area the degrees of settlement appear unrelated to fill thickness.

The most likely explanation for these phenomena is probably that the standard of initial compaction of the fill in close proximity to highwalls has a greater variance than say for the rest of the mine. The physical presence of the solid wall must also prevent lateral movements within the fill which can reduce in turn vertical displacements.

Collapse Settlement, Days 270-366:- An abnormal phase of settlement was found to have occurred on all stations over the fill during the period of 270-361 days from the start of monitoring. This phenomenon is detailed in Table 6. Settlement rates on either side of this period appear to be uniform and have continued in this uniform fashion to the present day. Of particular note is the fact that settlements on all stations over the fill are continuing at the rate of 10 mm/year, 9.5 years after monitoring commenced.

The occurrence of a sudden and dramatic increase in the rate of settlement on all stations sited over fill would appear to be akin to a phase of groundwater recovery. The fill in this area is shallow and it would be reasonable to suggest that groundwater recovery could have been completed in the period of

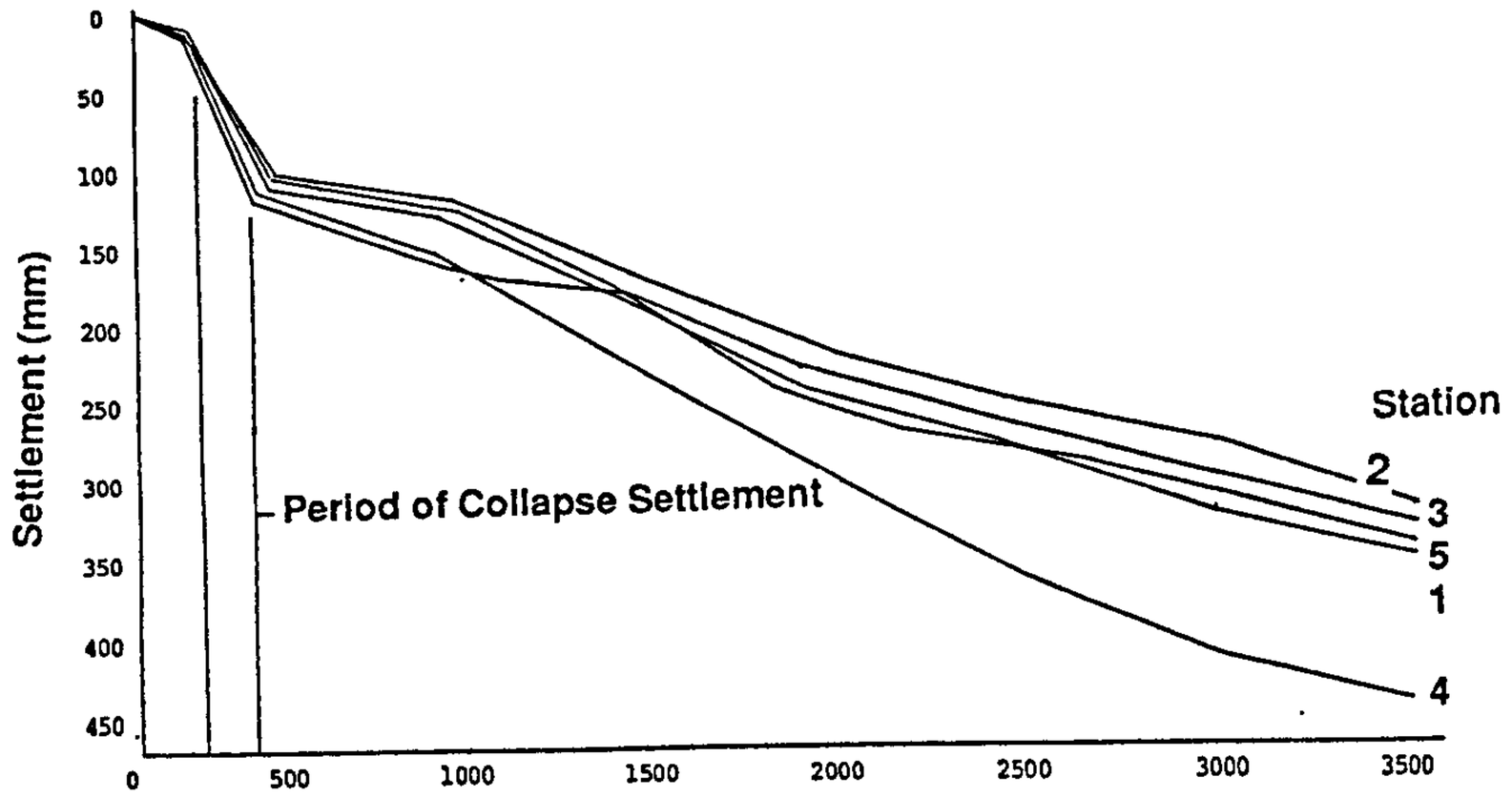

Figure 8. Settlement Results; Dragline Site C.

Table 5. Total Settlements, 1975-1985; Dragline Site C

Station	Approx. Depth of Fill (m)	Settlement (mm) (7.11.75 - 20.5.85)	Settlement as mm/m Fill Depth
1	34	-348	10.2
2	36.5	-288	7.9
3	15	-308	21.0
4	18	-451	25.1
5	12.5	-340	27.2
6	0	+3	-
7	0	-2	-
8	0	+3	-

+ denotes heaving
- denotes settlement

Table 6. Settlement Results, Days 0 - 452; Dragline Site C

Station	Time Elapse since Commencement of Monitoring (Days)				
	0 - 101	101 - 185	185 - 270	270 - 361	361 - 452
1	-7	-2	-	-93	-15
2	-9	-4	-1	-101	-13
3	-13	-7	-4	-103	-13
4	-12	-5	-2	-82	-9
5	-20	-10	-6	-76	-8
6	0	0	+1	-1	+1
7	+1	+1	0	+1	0
8	0	+2	0	0	+1

Displacements given in mm

91 days after day 270.

Figure 9 details the piezometer records for a piezometer on an adjacent opencast mine site, Figure 10 illustrates the standing water levels in the colliery pumping shaft.

The piezometer results show that in the period in question, the water levels rose by about 4 metres within this borehole to attain a level of 16.45 metres B.O.D. (below Ordnance Datum). The shaft water levels also show a rise in water levels which occurred over this period. These results would appear to leave it in no doubt that groundwater

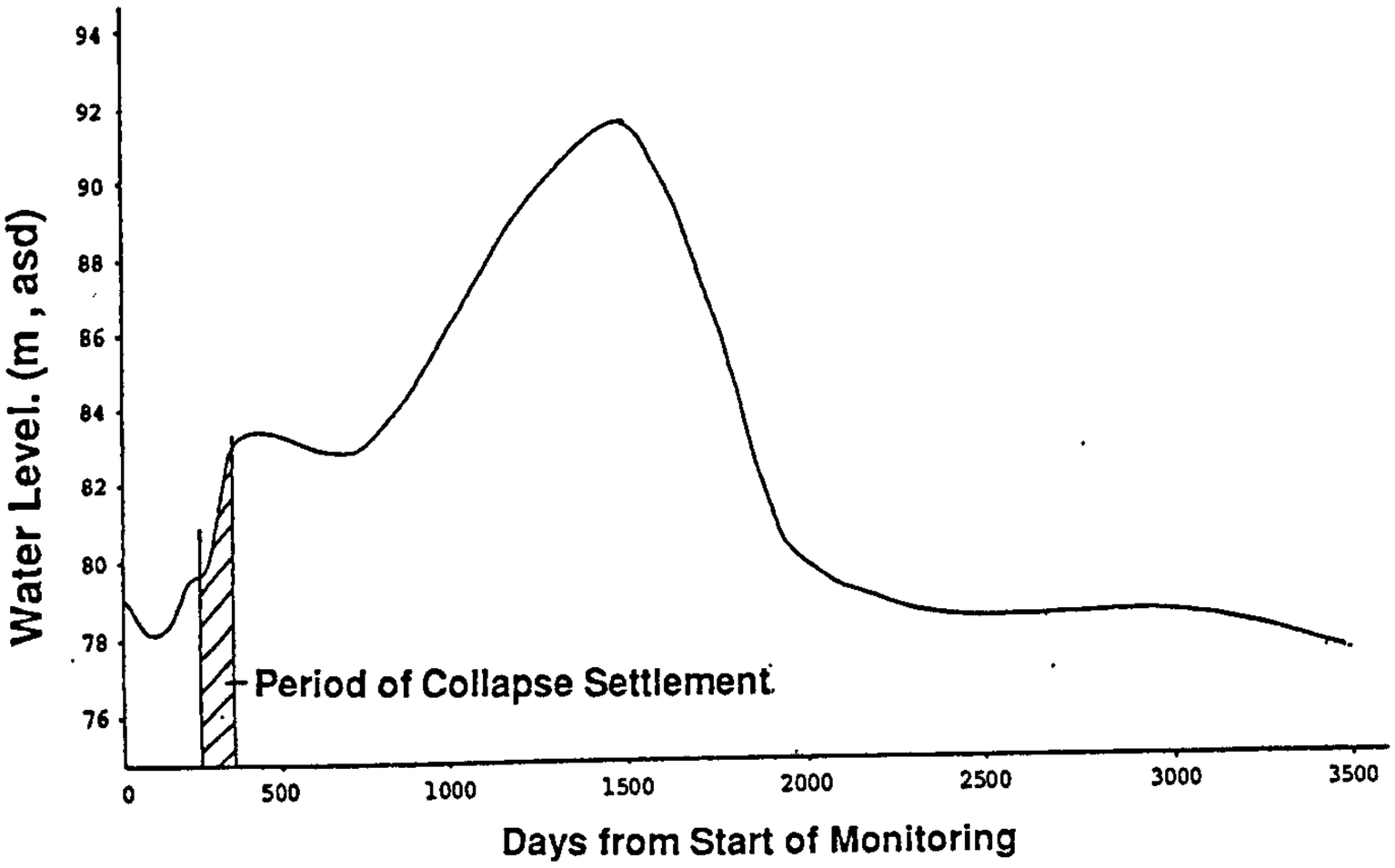

Figure 9. Water Levels in Local Piezometer; Dragline Site C.

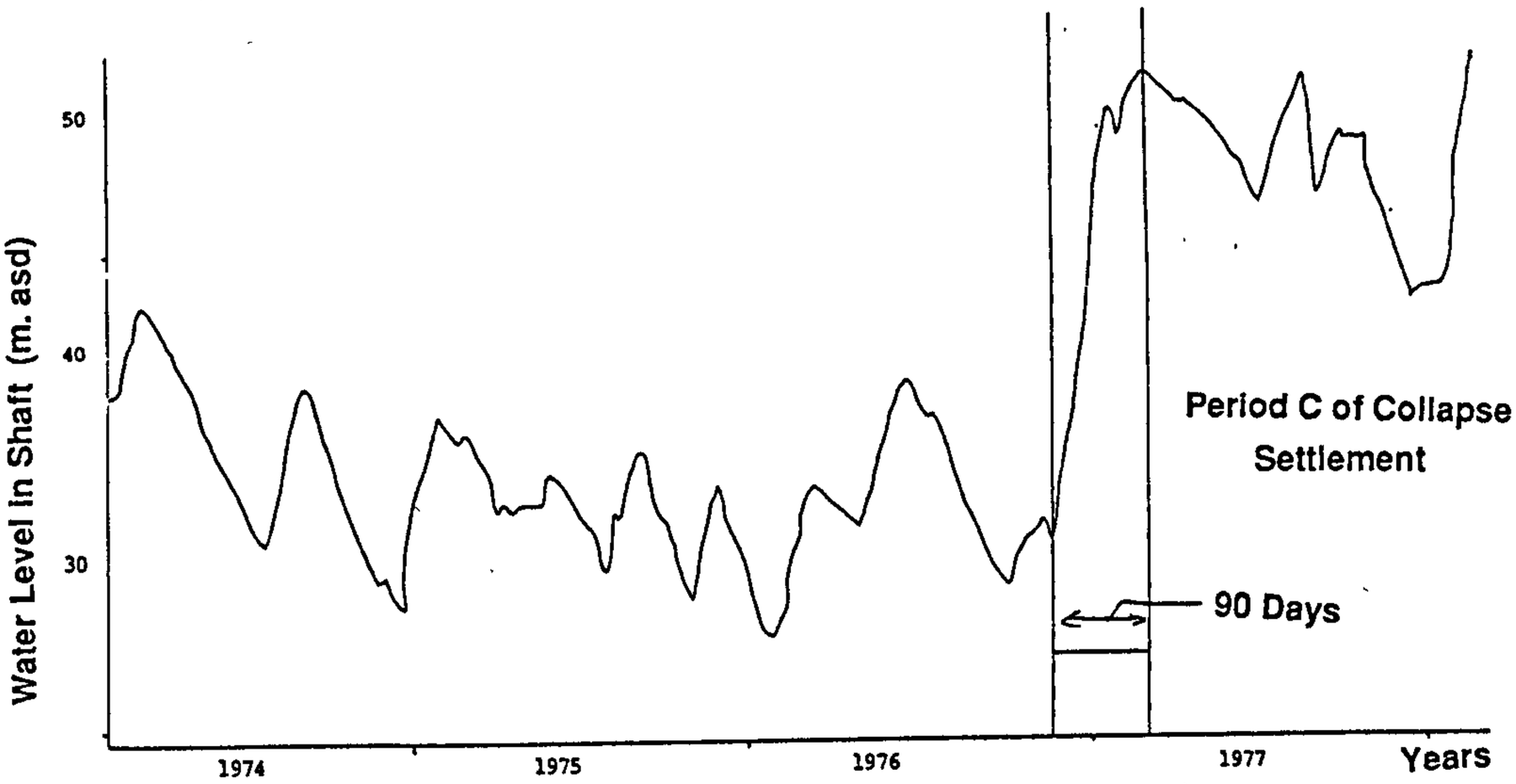

Figure 10. Water Levels in Pumping Colliery Shaft.

recovery occurred in the Dragline Site D fill to cause the increased settlement rates.

Following the large settlements of this period, a reduced settlement rate has continued in the fill for 8.5 years. This implies that even following the completion of groundwater recovery, significant movements can occur in shallow fills for a considerable time. It is unfortunate that this particular site was not instrumentated with magnetic extensometers and piezometers to observe whether subsequent movements have occurred in saturated or unsaturated fill materials.

TRUCK AND SHOVEL SITE D

The Site D was worked between 1972 and 1973 to win about one quarter of a million tonnes of coal and 4,000 tonnes of fireclay, by truck and shovel methods. The nature of the fill is 10% drift, 75% sandy shale and 15% sandstones. There were no difficulties with water during excavation.

Monitoring Scheme:- The monitoring scheme consisted of 8 surface levelling stations installed over a highwall which were tied into a temporary Ordnance Survey Bench Mark. The scheme is illustrated in Figure 11. Settlement observations commenced in late 1973 and continued to mid 1981. In addition to monitoring vertical displacements, changes in the coordinates of the stations were also recorded to derive a three dimensional view of backfill settlement. Horizontal Displacements were only recorded over the initial three years.

Results:- Settlement results are tabulated in Table 7. Differences in Northings and Eastings are presented along with total horizontal movements in Tables 8a and 8b.

The maximum settlement over the 8 year monitoring period was 48 mm. As no water was encountered during mining and there are no obvious settlement anomalies it is fair to state that this fill is unsaturated and

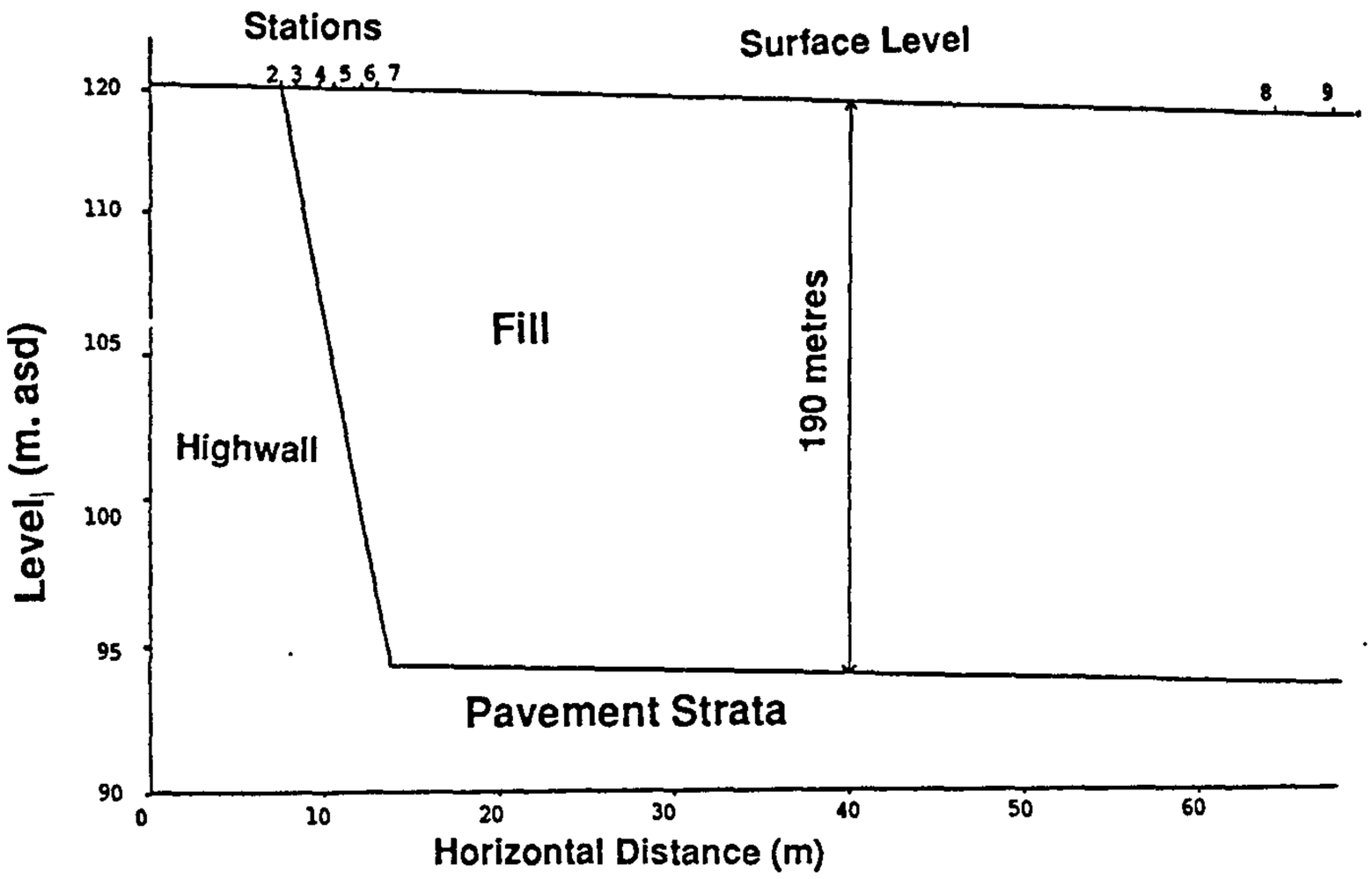

Figure 11. Instrumentation Scheme; Truck and Shovel Site D.

Table 7. Total Settlement; Truck and Shovel Site D

Surface Station	Total Settlement Recorded (mm) 1973 - 1981
2	0.25
3	1.9
4	5.6
5	13.6
6	23.6
7	26.1
8	44.9
9	48.5

Table 8a. Changes in Eastings, Surface Stations Site D

Date	Changes in Easting (mm)							
	Station							
	2	3	4	5	6	7	8	9
8.11.73	0	0	0	0	0	0	0	0
13.04.75	+18	0	-6	-11	-7	-2	-29	-23
11.06.75	+19	+18	+2	-3	+10	-10	-11	-26
13.04.76	+6	+1	-8	-21	-29	-28	-56	-48

that water percolation is solely due to that from surface infiltration.

Stations 6, 7, 8 and 9 are all sited in fill of similar depths, however twice the settlements of 6 and 7 have been recorded on 8 and 9. The presence of the highwall cannot be discounted from having effect however as for Dragline Site A, to critically "over-analyse" settlement results of 48 mm maximum in 19.6 m of fill could well be misleading.

Table 8b. Changes in Northings, Surface Stations Site D

Date	Changes in Northing (mm)							
	Station							
	2	3	4	5	6	7	8	9
8.11.73	0	0	0	0	0	0	0	0
13.04.75	-9	+6	+10	+7	+4	+8	+21	+15
11.06.75	-11	+1	+7	+1	-12	+5	+11	+5
13.04.76	-7	+6	+9	+5	+16	+9	+26	+24

Table 9. Total Horizontal Displacements, Site D

Station	2	3	4	5	6	7	8	9
Movement (mm)	9	6	12	22	33	30	62	54

Table 9 details the horizontal surface displacements for each station. Of great importance is to note that these horizontal movements are in general greater than vertical settlements experienced. Table 9 shows that station 8 has displaced 62 mm up to mid 1976 whilst only 29 mm vertically. If the construction of structures is proposed then it would be this horizontal movement that would be important and not necessarily the vertical. Differential horizontal surface strains is not a subject which has been considered in previous work on backfill settlements although it is well known in subsidence work. For construction purposes, however, it is essential that they be considered.

Table 10 details the surface movements over the site as ground strains between the largest being between Stations 6 and 7 of -26.7 mm/m generally manifest around the area of the highwall - the strains between Stations 7 and 8 being generally relatively low (0.54 - 0.5 mm/m). The strain values presented in Table 10 do show that strains between two stations can change in both magnitude and

Table 10. Lateral Strain Summary; Truck and Shovel, Site D

Interval	Distance (m)	Lateral Strains mm/m of Interval			
		8.11.73	10.04.75	11.06.75	13.04.76
2 - 3	1.0	0	-21.0	+16.5	-3.9
3 - 4	1.4	0	-5.3	-7.5	+5.3
4 - 5	1.0	0	+3.6	+0.9	-7.2
5 - 6	1.4	0	+3.3	+7.2	-17.0
6 - 7	1.0	0	+3.9	-26.7	+25.7
7 - 8	46.0	0	-0.6	+0.5	-0.6
8 - 9	3.0	0	+2.8	-6.9	+7.2

direction with time. Both compressive and tensile strains have been experienced on all the individual sections.

DRAGLINE SITE E RESTORED AS COUNTRY PARK

Site E, worked between 1966 and 1971, extracting 2.5 million tonnes of coal in a boulder clay, sandstone, fireclay and shale overburden, by dragline methods. The area was restored to form a country park which involved the construction of a lake with a concrete lined channel leading to the sea. Channel excavation commenced in April 1983 and its construction incorporating concrete base sections was completed by June/July 1983. The contractual maintenance period terminated on 26th October 1984.

In June 1985, it had been reported that a section of the culvert had sunk into the fill over a 45 m length. The settlement had been of the order of 1115 to 1160 mm.

This occurrence has been of great interest as it involves a settlement delay of around 2 years after the channel had been constructed over fill that was 14 years old. The fill depth is around 33 metres and thus this represents a settlement of 3.4% expressed in terms of fill thicknesses - a much larger settlement than experienced in earlier reported observations.

Adjacent to the subsided section of the channel four boreholes were drilled through the full depth of opencast backfill and logged using geophysical (slimline) techniques; also a piezometer was installed in one borehole. This investigation indicated the existence of a high proportion of voids in the lower layers of the backfill, and that the materials at these (dragline cast) levels was predominantly sandstone. The piezometer indicated that no water table existed in the backfill.

It is conjectured that this local subsidence of the channel resulted from some form of isolated and delayed settlement mechanism in the lower backfill layers. No leakage from the channel occurred.

TRUCK AND SHOVEL SITE F RESTORED AS A PLAYING FIELD

Site F worked by truck and shovel methods to an average depth of about 11 metres. Mining terminated in 1961, and the surface of the site now acts as a playing field for the local school.

Attention has been recently drawn to the fact that there are three or four surface depressions of irregular shape that have appeared on the fill surface. The largest of these depressions being 500 m^2 in area with a maximum depth of around 0.5 m. The depressions have only become evident within the last few years. These movements are of interest for the following reasons:

- The fill has been in position for 24 years
- A settlement of 0.5 m in 11 m of fill represents 4.5% subsidence
- Inspection shows no signs of current or previous slope failures or water issues
- The last date of deep mine working was 1945, 80 m below the surface.

Akin to the observations from dragline site E there appears to have occurred some exceptionally long-term settlement movements. No detailed study has been conducted and any remedial work is likely to comprise of the infilling of the surface troughs with imported material.

CONCLUSIONS

(i) At Dragline Site A, fill movements, after 7 years of monitoring, were found to be negligible. A fluctuating groundwater table was found no longer to induce collapse settlements.

(ii) Groundwater associated settlement was identified at Dragline Site B. On completion of groundwater recovery there was a reduction in the rate of settlement. After a seasonal lowering of the groundwater, secondary recovery re-activated collapse settlement in previously saturated fill.

(iii) Groundwater associated settlement at Dragline Site C was identified. Settlement magnitudes in general were not considered a function of fill depth, more the degree of saturation. Settlement magnitudes were affected by presence of adjacent highwall, preventing lateral fill movement. Water recovery commenced in the fill 270 days after monitoring and continued for 90 days. Since completion of recovery, the settlement rate has remained steady for 9 years at 10 mm/year.

(iv) Observations at Truck and Shovel Site D emphasised the importance of lateral movement prediction. Surface lateral movements exceeded total vertical settlements and were found to continue for at least three years on this site. The absence of fill saturation is considered the most important factor in the lack of total vertical settlement.

(v) A Dragline Site E settlement of 3.4% was recorded in a fill 14 years old, two years after the construction of a concrete lined channel. The settlement area occurred over a 40 m length of the culvert. It is presumed a result of a delayed settlement phenomena in the lower layers of fill.

(vi) At Truck and Shovel Site F a 4.5% settlement was recorded in a backfilled site in 24 years. It is unknown over what time scale these movements have occurred. As the site comprised part of a hillside, the land was free draining and not liable to groundwater recovery although infiltration rates would have been significant.

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PAPER C12 NORTHERN BACKFILL 4

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Long term settlement of opencast mine backfills – case studies from the North East of England.

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Long term settlement of opencast mine backfills – Case studies from the North East of England

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ABSTRACT: The paper presents the results of four investigations relating to long term uncompacted backfill settlement in the North East of England. The research which covers a period from 1974 to 1988 is aimed at providing a better understanding of the complex deformation processes which backfill materials are subject to, and consequently contribute to the design of compacted backfill placement for restored sites destined for structural development. The results indicate the interaction of mine geometry, fill properties, method of working and groundwater on long term deformations.

1. INTRODUCTION

Observations related to backfill settlement in the North East of England commenced in the 1960's, when Kilkenny, (1968), investigated the suitability of restored opencast mine sites for structural development. A detailed investigation using surface levelling stations and borehole instruments was conducted on the Horsley site monitoring the effects of groundwater recovery and the related induced collapse settlement of backfill materials, Charles et al (1977), (1984). In 1983, British Coal Opencast Executive commissioned research conducted by Nottingham University, Department of Mining Engineering into groundwater recovery problems associated with opencast mine backfills, Reed, (1986). This project which was conducted on a national basis of field work received a substantial input from monitored sites in the North East of England, and presented results of monitoring from 1975 to 1986 on a variety of sites. This paper extends the findings of this work.

1.1 Uncompacted and compacted backfills

The results presented in this paper are applicable to backfill masses which have been conventionally replaced by plant without specific compaction treatment. Where structural development of a site is known prior to operations, e.g. for road construction or light structural development, then the backfill is compacted in accordance with a designed specification. The specification ensures that future construction is not adversely affected by movements in the foundation fill. The behaviour of uncompacted fills is however important for the following reasons;

a). Development of an uncompacted site several years after completion which was not previously envisaged at site design stage.

b). Construction of services, water courses etc. on fills not warranting a high degree of compaction.

c). Understanding of the settlement properties of uncompacted fills is a prerequisite to the efficient design of a compaction operation.

1.2 Factors affecting the rate and magnitude of backfill settlements

The most commonly recognised factors affecting the settlement process of an uncompacted backfill are as follows;

a). Method of fill placement.
b). Fill materials
c). Influence of groundwater.
d). Mine geometry.

Broken Coal Measures rock strata can exhibit considerable volume changes when

used as a backfilling material. Silts and plastic clays expand when wet, whilst soft organic materials such as peat can compress on dumping. It is where bulkage is high that the possibility of significant settlement exists. In the course of normal operations, mining machinery can give complimentary albeit indiscriminate compaction to fill materials. The initial bulkage is related to initial particle size, which in turn can be dictated by mining method. End-tipping by dump trucks or dragline spoiling can lead to a gradation of particle sizes with depth in the fill. The larger blocks of material will tend to fall further with gravity and consequently the lower layers of a fill may be significantly less dense than nearer surface. The use of scraper plant gives greater compaction in its method of layer replacement and by the fact that the smaller blocks handled produced a fill of greater initial density. The advent of large hydraulic shovels has also enabled larger block volumes of overburden to be excavated with consequently increased bulkage.

Traditional settlement theory postulates a logarithmic decay of fill settlements with time. This however assumes the self weight of the fill to be the controlling mechanism and does not consider additional effects such as the impact of water table fluctuations on fill stability. The stresses exerted by the action of water can have a great effect on the individual blocks or particles within a backfill mass, owing to the ability of water to weaken rock strength. If the rock strength is considered to be proportional to its surface energy, then this is reduced in the presence of water, with a resulting reduction in compressive strength, figure 1.

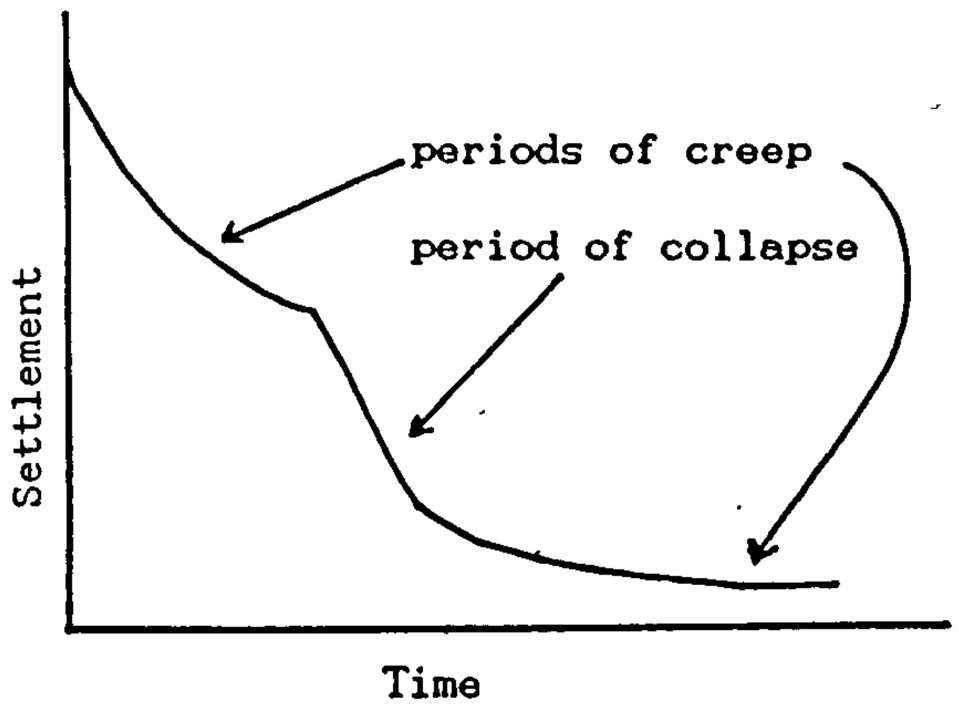


Fig. 1 Traditional Settlement theory

A water table which rises gradually within a fill, either as a consequence of natural recovery of levels after mining or by external influences, (e.g. cessation at adjacent pumping sites or of deep mine pumping), will have an affect of accelerating the physical and chemical degradation of fill materials. Several authors have produced research to show the close correlation between collapse, (or sudden accelerated settlement), and fluctuations in the water table within the fill. The following case studies demonstrate the effect of water recovery on such backfill movements, and indicate the timescales which may elapse prior to sudden collapse settlements superimposed on conventional creep movements.

2 INTRODUCTION TO CASE STUDIES

The four case studies, (Radcliffe, Coldrife, Radar and Sisters sites), relate to opencast mines in a relatively small area of Northumberland close to the coast. The positions of these mines is indicated in figure 2. The geology of the area is typically Coal Measure mudstones, sandstones, coals and seatearths under a covering of glacial drift materials up to 30 m thick. A common ratio of overburden excavated on a site to coal won is 20:1. Typical working methods for sites involve loose tipping draglines, shovel and truck, as well as motor scraper plant. The groundwater over much of the area is affected by regional deep well pumping immediately adjacent to the former Hauxley Colliery shaft and by nearby active opencast workings.

3 CASE STUDY 1. RADCLIFFE SITE

3.1 Observations

Radcliffe site worked from 1971 to 1976. The instrumentation scheme which consisted of surface levelling stations commenced in 1975 over a traverse crossing an excavated and backfilled slope as detailed in figure 3. The results of the monitoring to date is presented in figure 4.

The following observations can be made from inspection of the above two figures;

a). Settlement profiles are similar for all stations, showing;

- o A slow period of settlement from late 1974 to mid 1976.
- o A period of collapse settlement

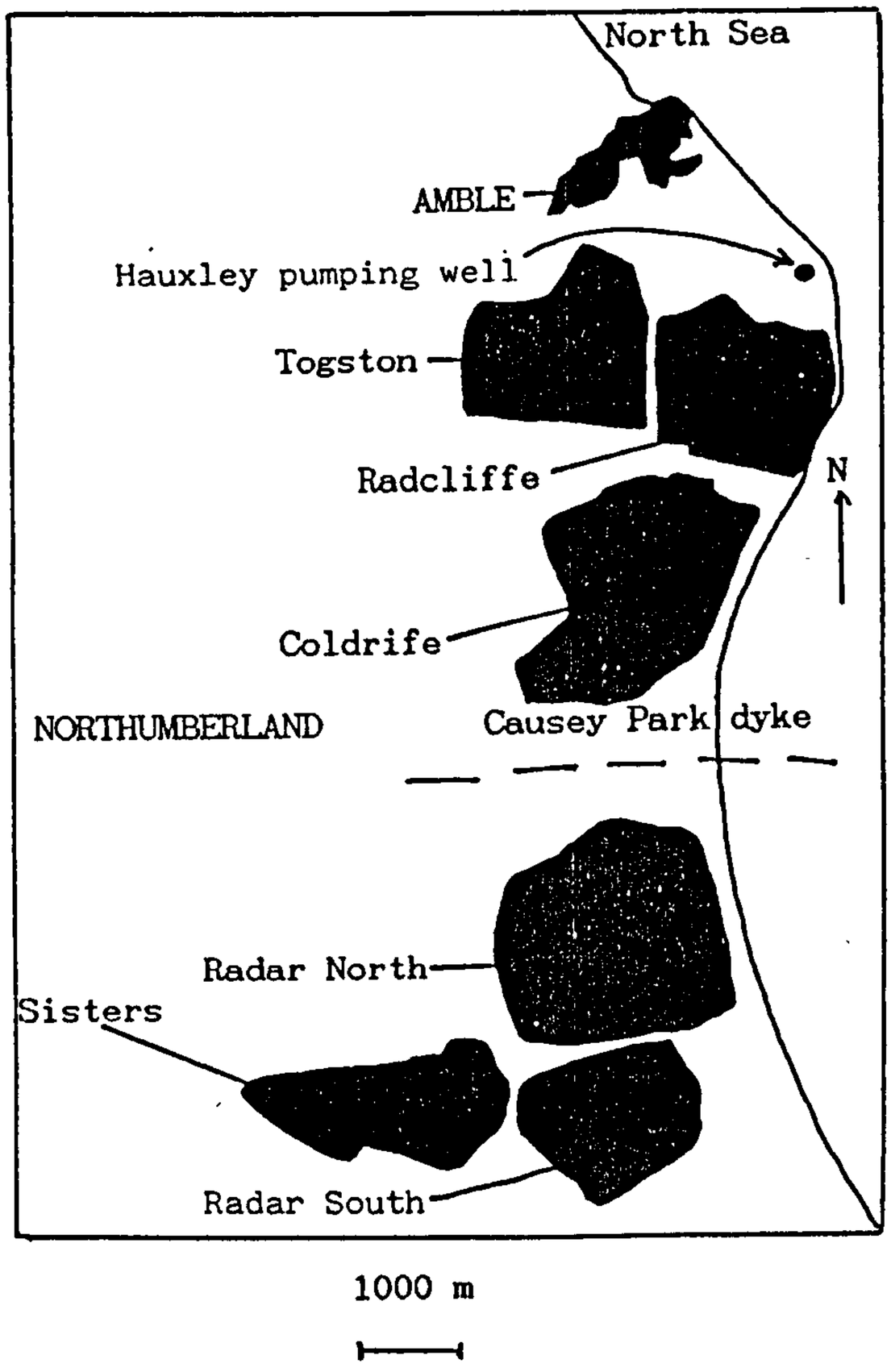


Fig. 2. Site location plan

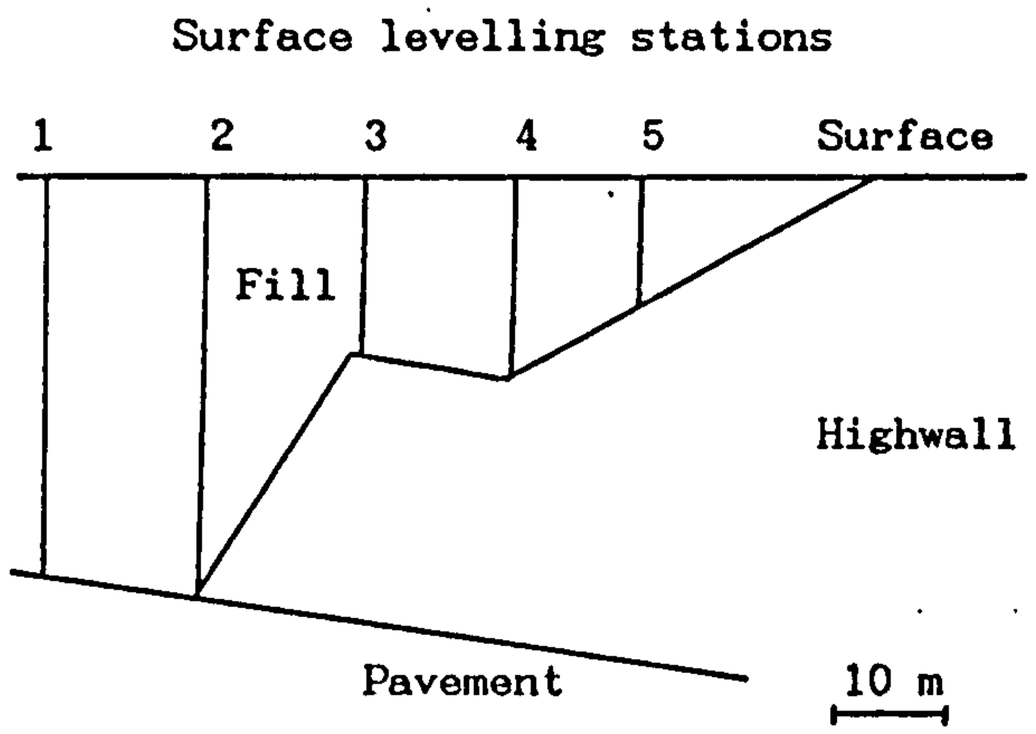


Fig. 3 Radcliffe site. Monitoring scheme.

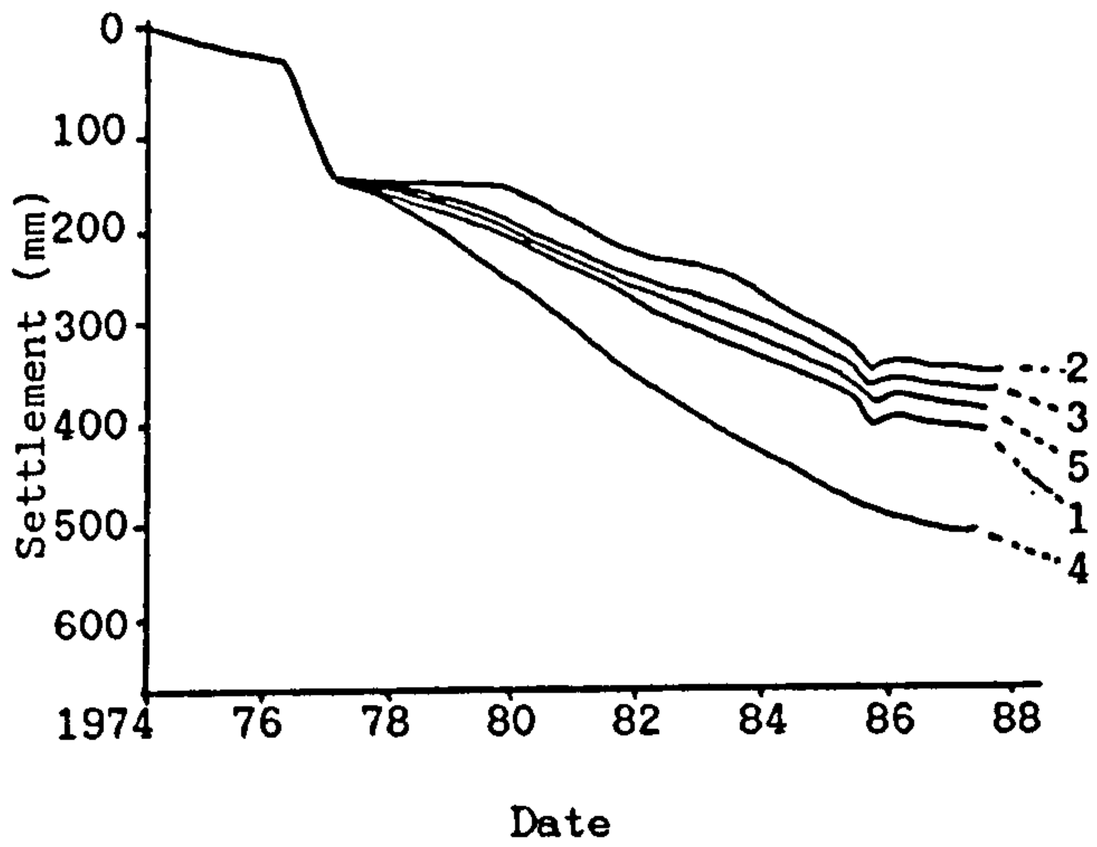


Fig. 4 Settlement results, Radcliffe site

between mid 1976 to late 1976.

o Continued long term settlement from late 1976 to mid 1985.

o A small period of collapse settlement from mid 1985 to early 1986.

o A period of settlement recovery (heave) followed by continued settlement from early 1986 to mid 1986.

o Continued long term settlements to mid 1987.

b). The magnitudes of settlements are not related to fill depth. The magnitudes of backfill settlement are up to 3% of the fill depth.

Table 1 details magnitudes of settlement relevant to the study.

No piezometers have been retained actually within the Radcliffe site boundaries, however fluctuations in water levels can be inferred from piezometers on nearby sites and water levels recorded in the Hauxley Colliery shaft over the period of monitoring.

Total settlement 1975 - mid 1987 .

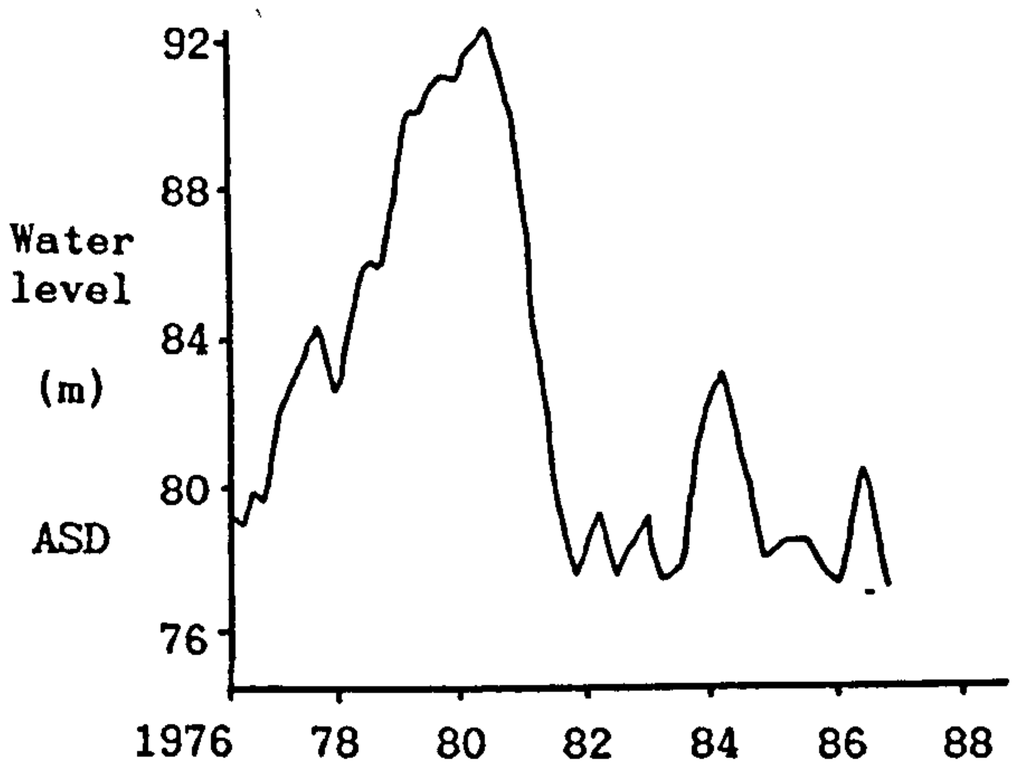
Stn	Fill Depth (m)	Settlement (mm)	Settlement %fill depth
1	34	399	1.17
2	36.5	332	0.91
3	15	348	2.32
4	18	504	2.80
5	12.5	373	2.98
6	0	+2	-
7	0	+6	-
8	0	+3	-

Table 1 Settlement; Radcliffe site

Figures 5 and 6 show water levels from 1975 to mid 1985 for the colliery shaft

ASD; above site datum = 100 m below ordnance datum

Figure 5. Water levels recorded in Hauxley Colliery pumping well



ASD = above site datum = 100 above OD.

Fig. 6 Water levels in piezometer, Togston site

some 1000m distant and a piezometer on the adjacent Togston site 2000 m distant. The influence of the Hauxley pumping can be seen to have significant impact on the water levels on the Togston site and hence may be used to subjectively consider water levels affecting the monitoring exercise on the Radcliffe site.

Water movements in the colliery shaft controlled by well pumping operations can be summarised as follows;

o Minor fluctuations during 1975 and 1976.

o Rising water levels, initially at a high rising rate from the beginning of 1977 to April 1978. Then a steady climb of water levels to around March 1980

o A sharp drop in water levels then occurred between from the end of 1980 to April 1981.

o From April 1981 onwards, occasional fluctuations in water level with a prominent peak from April 1983 to July 1984.

The collapse settlement in the Radcliffe fill occurring between August 1976 and November 1976. In this period the water levels in the shaft were falling until October 1976 when the general period of rising water levels commenced. With reference to the piezometer on the adjacent Togston site, water levels commenced to rise from mid 1976 onwards to early 1980. It is apparent that these rising water trends are also applicable to the Radcliffe site, however the sudden collapse settlement was only restricted to a few months rather than the approximate three years of recovery recorded on the Togston piezometer.

3.2 Collapse versus creep settlement

Of more importance than the period of collapse settlement on the Radcliffe site, is the extensive creep settlement that followed. To the last date of monitoring available, it is apparent that the fill has been settling since the period of collapse at more or less a uniform rate. The period since the completion of collapse settlement is of the order of 10.5 years. The uniform rate, (of up to 38 mm/year on station 4), has been slightly disturbed by a

sudden collapse movement towards the end of 1976. On this occasion, collapse settlement was noted on all stations except station 5 situated in the shallowest fill of 12.5 m deep. (Closest to the highwall). The period is associated with rise of water levels, however as figures 4 and 5 show, other water level rises previous to this occasion and after the initial period of collapse settlement did not result in further collapse settlement of the fill. The most significant comment lies in the fact that the fill was capable of undergoing a significant further collapse settlement some 10 years after an initial period of collapse settlement, and after a prolonged period of creep settlement.

3.3 Magnitude of settlements

Previously mentioned was the fact that the overall magnitude of settlements recorded on each station was independent of depth. This is particularly relevant for station 4 situated within a bench of the highwall. The most likely reasons for this are twofold;

- o The degree of compaction afforded to the fill in close proximity to the excavated wall may be reduced as plant have difficult access to the interface of the highwall and the backfill.
- o The interface between the solid and backfill may act as a conduit for surface water, thus lubricating the fill in this area more than in the main body of the fill, e.g. station 1, this resulting in an increased rate of settlement occurring through gradual infiltration and percolation of rainfall/runoff.

A third reason may be that there were significant material differences between the fill in this area to that in other parts of the traverse. This cannot be proved, other than by site investigation, and is most likely to be insignificant as compared with the two hypotheses postulated above.

3.4 Summary of findings; Radcliffe site

The following summary observations can be made from the investigation on the Radcliffe site.

- o The potential effect of long term settlements, (in this case over 10 years).
- o The inter-relationship between ground water fluctuations and collapse backfill settlement.

4 CASE STUDY 2. COLDRIFE SITE

The site worked from 1966 to 1971, and was restored to form a country park. The restoration involved the construction of a lake with a concrete lined channel leading to the sea. Channel excavation was commenced in April 1983 and the installation was completed by July 1983. In June 1985, it was reported that a 45 m length of the channel had subsided. The channel and subsidence zone is illustrated in figure 7, along with the working limits of the two opencast seams. The occurrence involves delayed settlement within a fill, approximately 30 m deep some 14 years old.

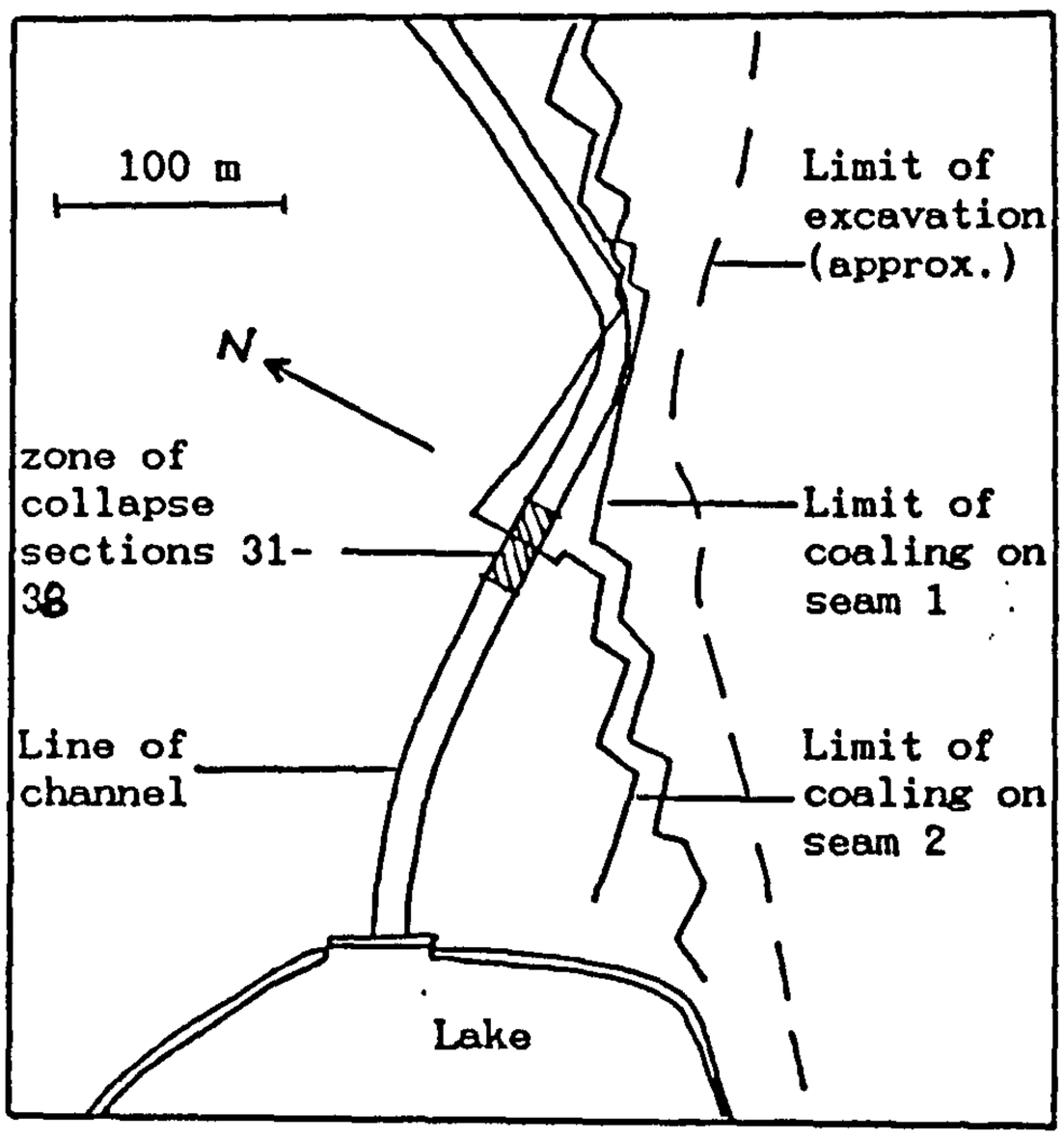


Fig. 7 Coldrife site, area of subsidence

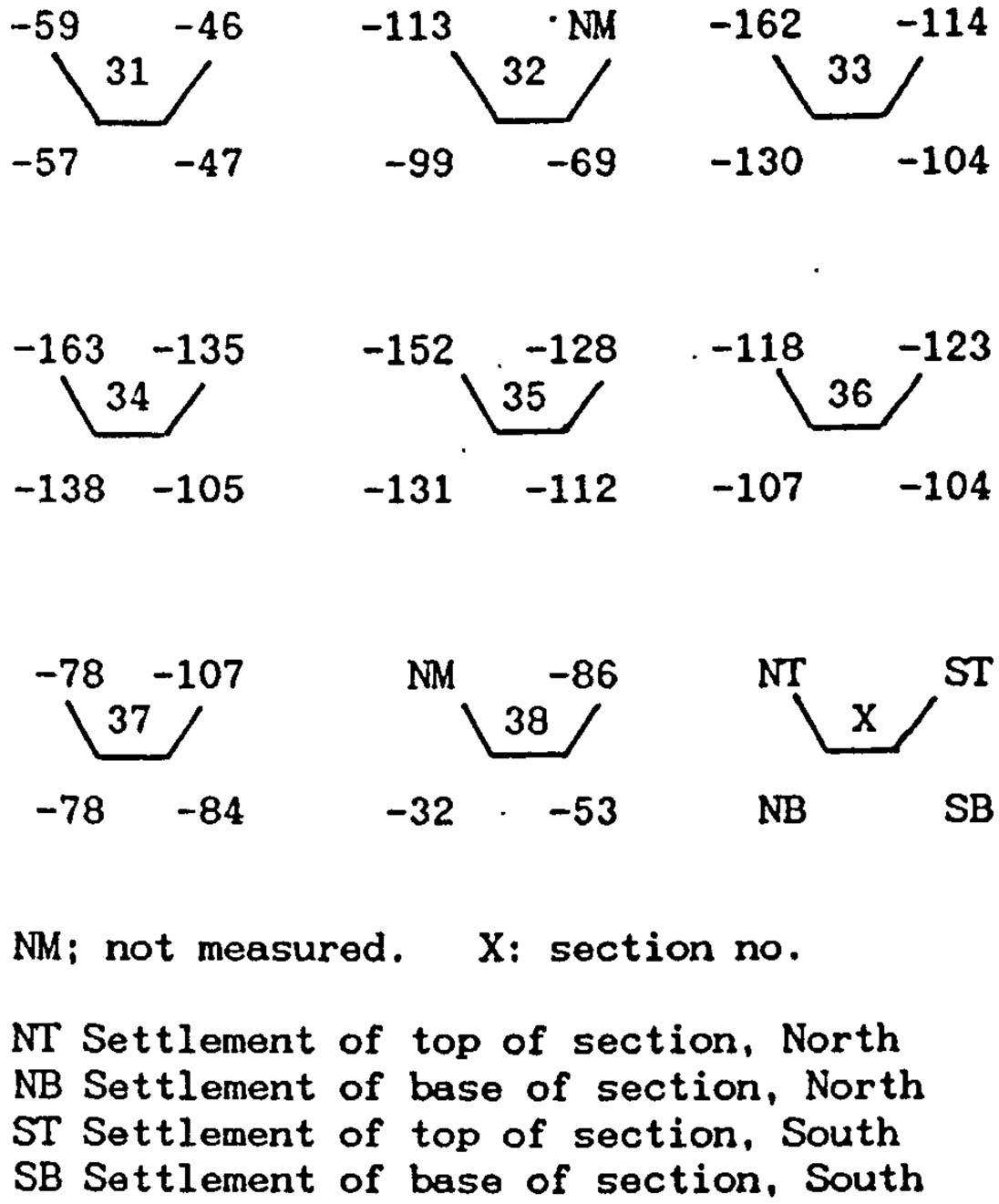
4.1 Investigation

Sections of the concrete channel had been periodically levelled prior to and after the collapse. In addition to this monitoring, after the collapse of the channel, three boreholes were drilled and the nature of the backfill investigated. A piezometer was installed in one of these boreholes.

4.2 Magnitude of settlement

The sections levelled are situated at 5.5 m intervals along the line of the channel. Figure 8 details the total magnitudes of settlement over the available monitoring

period. Figure 9 details the settlement records against time for the section of greatest subsidence, number 34. These curves are representative of the other sections in the collapse area.



NM; not measured. X: section no.

NT Settlement of top of section, North
NB Settlement of base of section, North
ST Settlement of top of section, South
SB Settlement of base of section, South

Settlement in mm.

Fig 8. Maximum recorded total settlements Coldrife site, 11.9.84-12.5.87

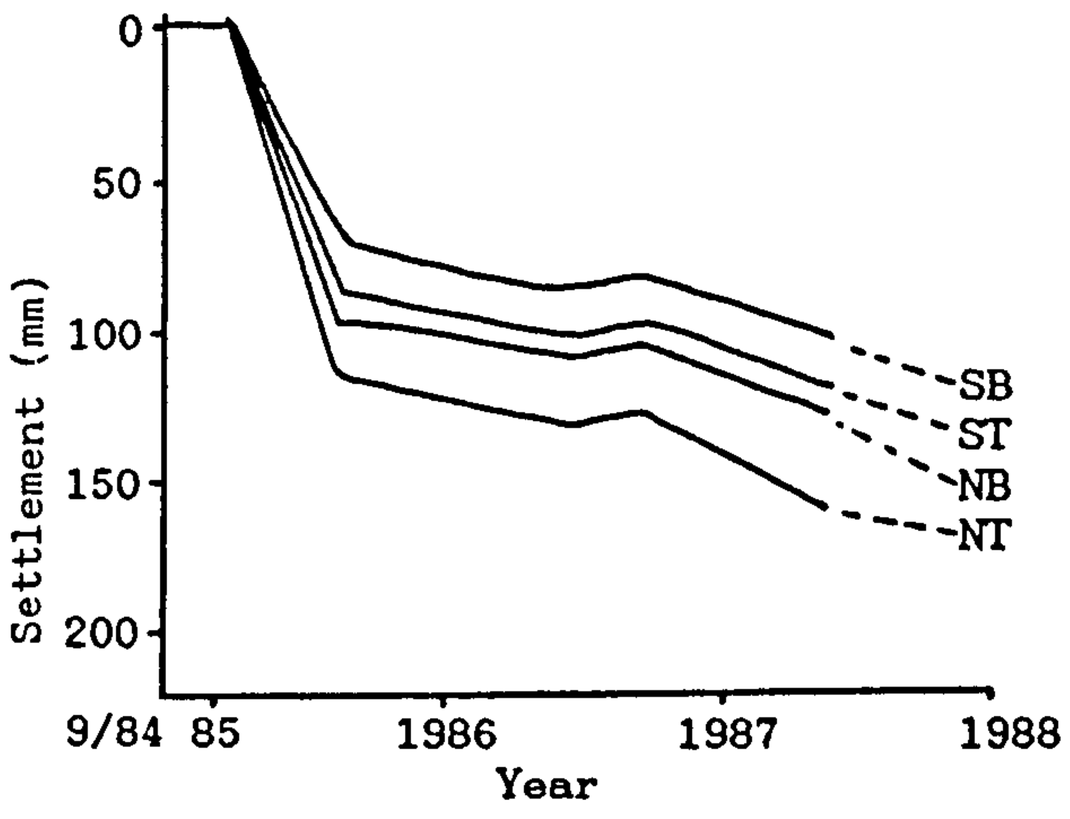


Fig. 9 Subsidence in section 34, Coldrife site

Examination of the above movements show the general slumping and tilt of the channel towards the height of the depression recorded on sections 33 and 34. Some degree of rotational movement is apparent. Records of settlement on either side of the collapsed zone indicate magnitudes of the order of 10 mm movement.

4.3 Groundwater behaviour

The behaviour of the groundwater on this site is again controlled to a large extent by the Hauxley pumping, but also by dewatering operations on the East Chevington site to the South. Reference is made to figure 5 in Case study 1.

The nature of the water within the fill was subsequently investigated by piezometer installation adjacent to the channel. The piezometer was installed to 4 m below the backfill pavement, acting purely as a stand-pipe within the fill. Results taken over the period of late 1985 to late 1989 show an almost constant water level of approximately 5 metres above the base of the backfill. Inspection of the water levels within the shaft and on the Togston piezometer show for the period of collapse settlement, (between November 1984 and June 1985), that water levels were dropping after a peak recorded around early 1984. With the Coldrife site being at a greater distance from the Hauxley shaft than the Togston site, it may be that the subsidence was related to this peak in water levels, which had taken a longer time to manifest within the Coldrife fill owing to the distance between the site and the control of groundwater. Previous peaks in water levels prior to channel construction indicate that collapse settlements may have actually occurred previously within the fill, but were not recorded.

4.4 Subsurface investigation

Three boreholes drilled to investigate the subsurface conditions were also gamma ray logged. Interpretation of the logs enabled the following conclusions to be drawn;

- o Water levels identified as commensurate with piezometer readings.
- o Broken mudstone and sandstone fill materials with greatest degree of cavitation towards the base of the fill. Occasionally cavities are described as large.

4.5 Summary

This case study complements that of the Radcliffe site, by indicating that

significant long term backfill movement can occur in uncompacted fills. Also demonstrated is the effect of water recovery on fills of this age can still result in collapse settlement. The scenario of collapse or increased settlement rates close to the interface beween fill and the excavated rock slopes of the sites is also apparent, as in the Radcliffe case. Whilst this is not purely a differential settlement at the fill/rock interface, it is an indication that backfill is naturally less compacted in such locations. The observation that the backfill contains a greater proportion of cavities towards its base, is again a reflection of the loose tipping method of working in conjunction with the difficulty associated with backfilling close to the excavation boundary.

5 CASE STUDY 3. RADAR SITE

The site worked between 1957 and 1973. On completion of backfilling, three surface levelling traverses were installed. Figure 10 details the layout of the traverses. The period of monitoring has been from the end of 1975 until mid 1987. Settlement movements can be related to a piezometer close to the excavation.

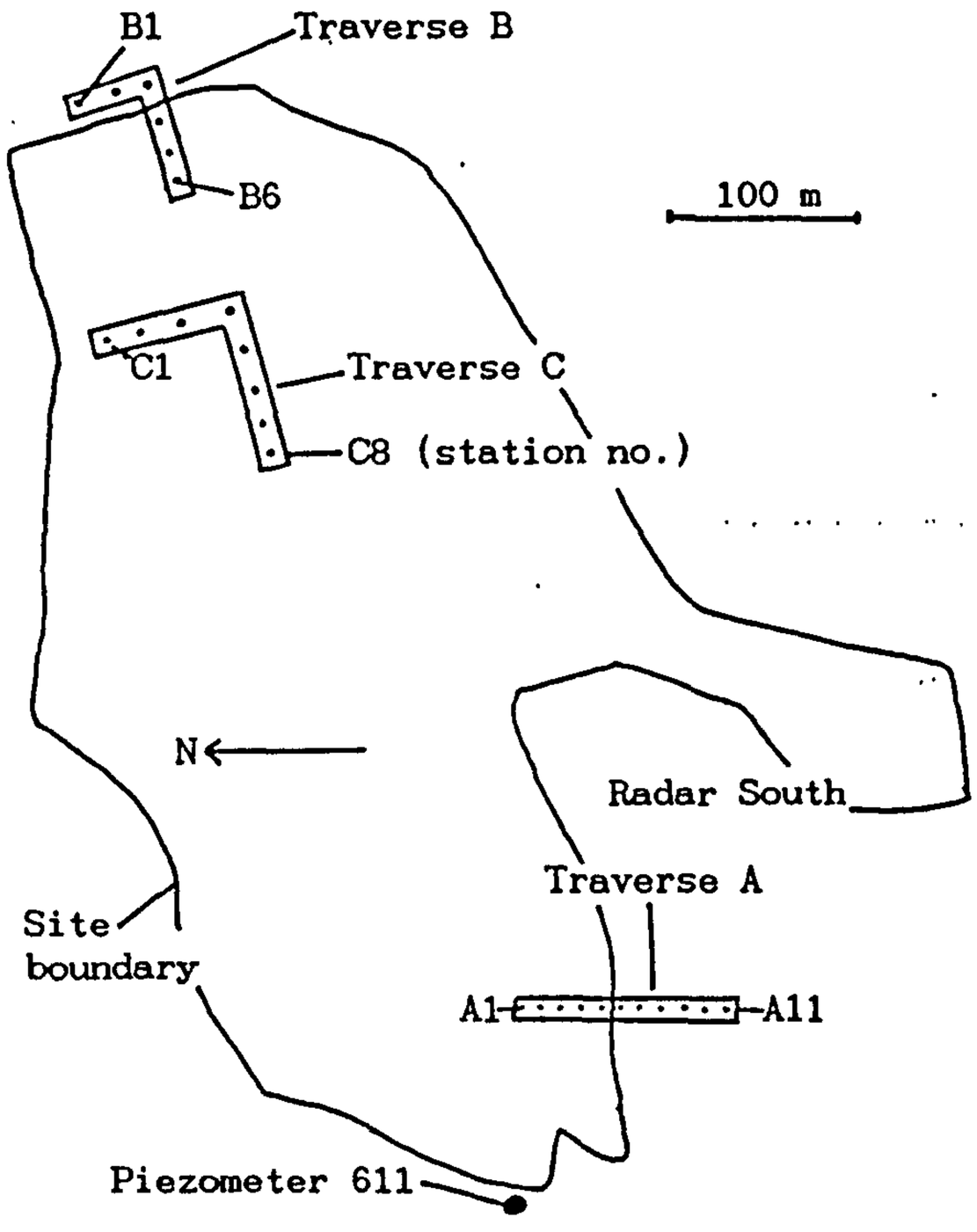
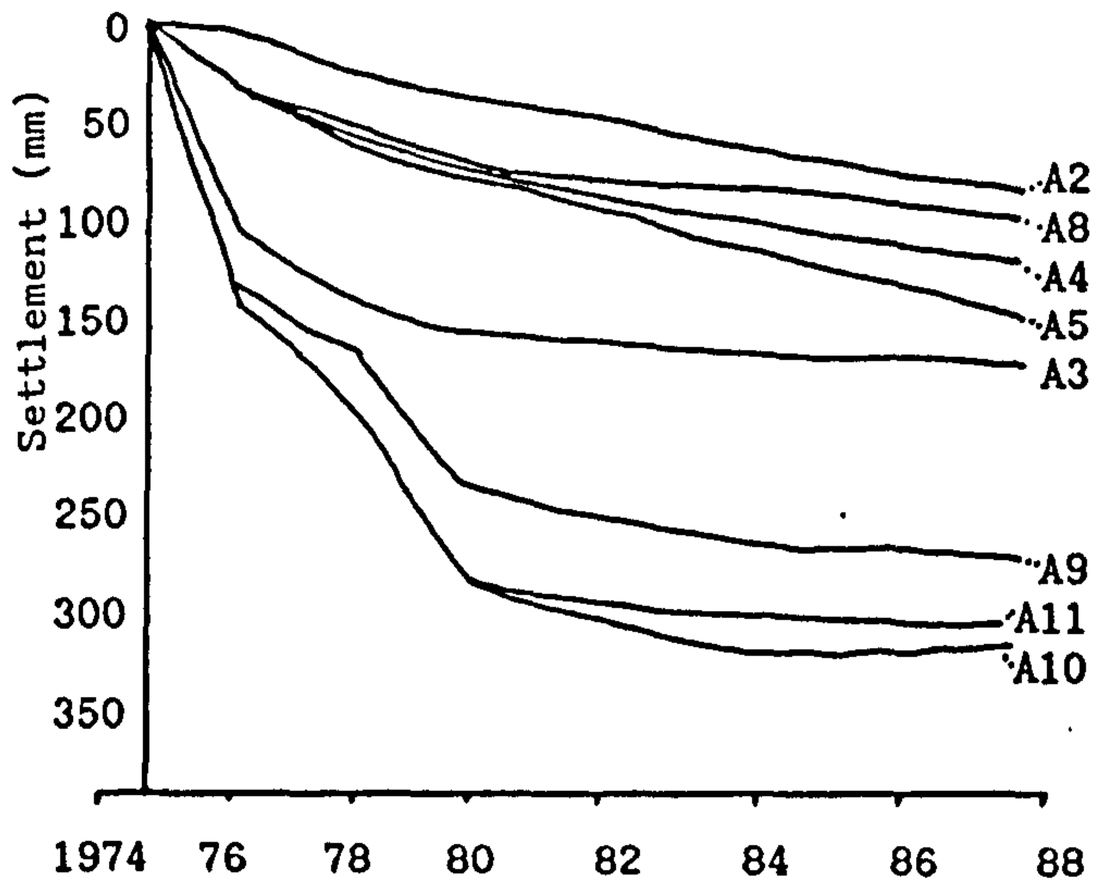


Fig. 10 Levelling traverses, Radar North

5.1 Traverse A

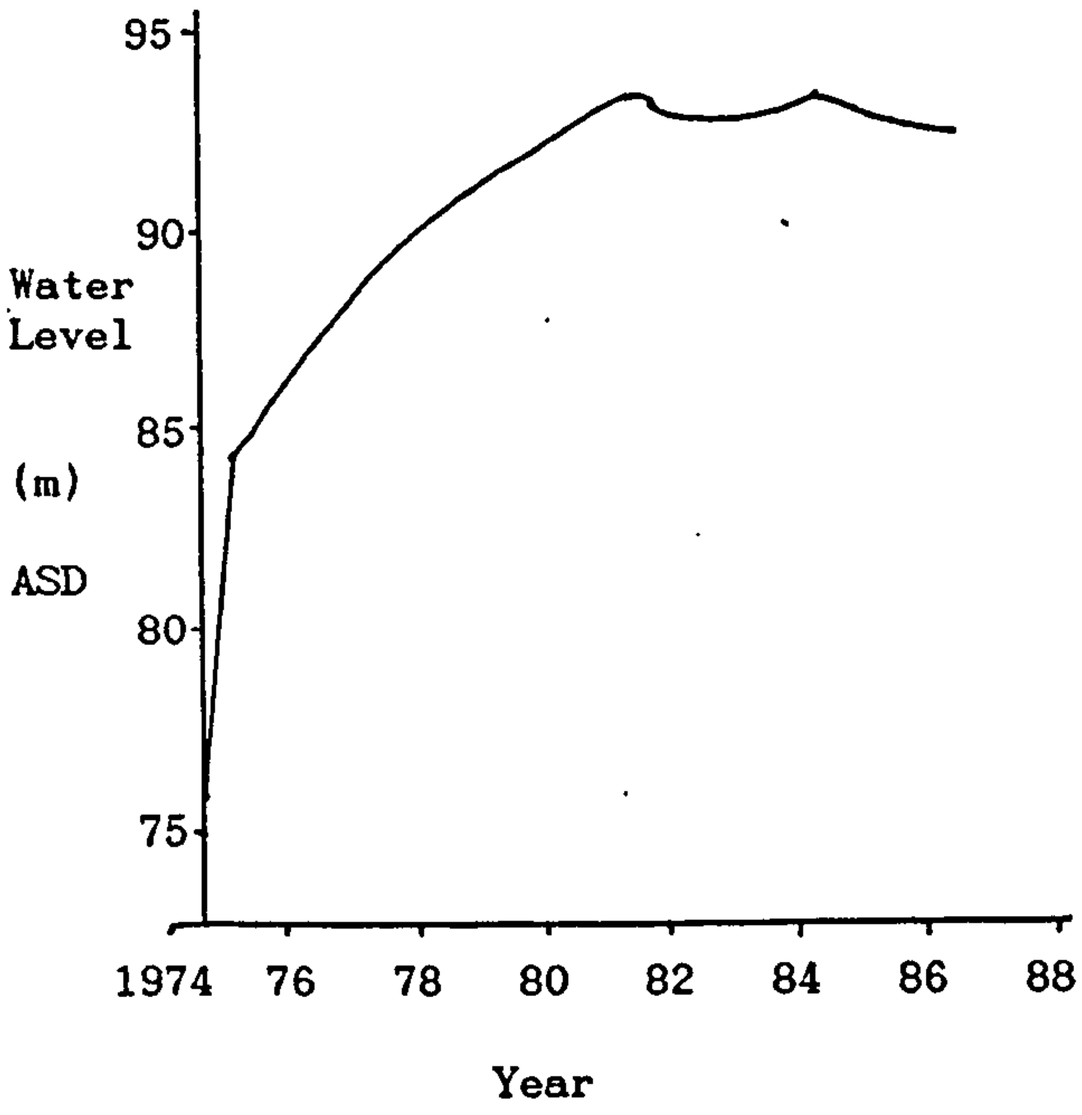
Traverse A passes over both the Radar site and the previously mined Radar South site which completed coaling in 1956. Settlement curves and water levels are presented in figures 11 and 12. The water levels from piezometer 611 indicates that the general trend of water recovery continued from the start of monitoring until approximately mid 1981. After this period only minor fluctuations in water level were observed. In this particular area the effects of dewatering at Hauxley can be discounted owing to the presence of an impermeable dyke to the North of the site. Total settlements recorded on the traverse are recorded in table 2.

The settlement curves thus show a good correlation between the rate of water recovery and settlement of the fill, although the settlement curves themselves appear to have a more classical logarithmic decay curve, rather than a sudden collapse in level associated with a sudden groundwater rise. The slow steady recovery of groundwater within the fill may explain this phenomenon. Settlement magnitudes are again not directly related to fill depth. Stations A2 and A3 are ones situated in the newest fill of the traverse and thus greater magnitudes of settlement have been recorded as might be expected.



Refer table 2 for total settlements.

Fig. 11 Radar North, Traverse A, settlement results, 1974 - 1988



ASD; above site datum = 100 m below Ordnance datum

Fig. 12 Piezometer water levels, No. 611 Radar North site

Traverse A, Monitoring from May 1974

Stn	Settlement mid1981 (mm)	Settlement mid1987 (mm)	Fill Depth (m)	Settlement %fill depth
A1	+6	+27	6	-
A2	-58	-89	7	1.27
A3	-172	-174	7	2.49
A4	-91	-123	18	0.68
A5	-111	-157	35	0.95
A6	-11	-6	boundary of two sites	
A7	+4	+15		
A8	-85	-102	21	0.49
A9	-261	-272	24	1.62
A10	-308	-322	21	1.53
A11	-300	-309	18	1.72

Table 2. Magnitudes of backfill settlement; Radar Site

5.2 Traverse B

Settlement results for traverse B are presented in figure 13 and table 3. The traverse location on figure 10 indicates that three of the stations were sited on solid ground, the traverse crossing a highwall with three stations on fill. The trend of the instrument movements are similar to those recorded on traverse A, although a period of monitoring was omitted between 1976 and 1983, resulting in a significant loss of information. Since 1983 however there has been very little movement on the stations, and this may be taken to correspond with the groundwater observations on piezometer 611.

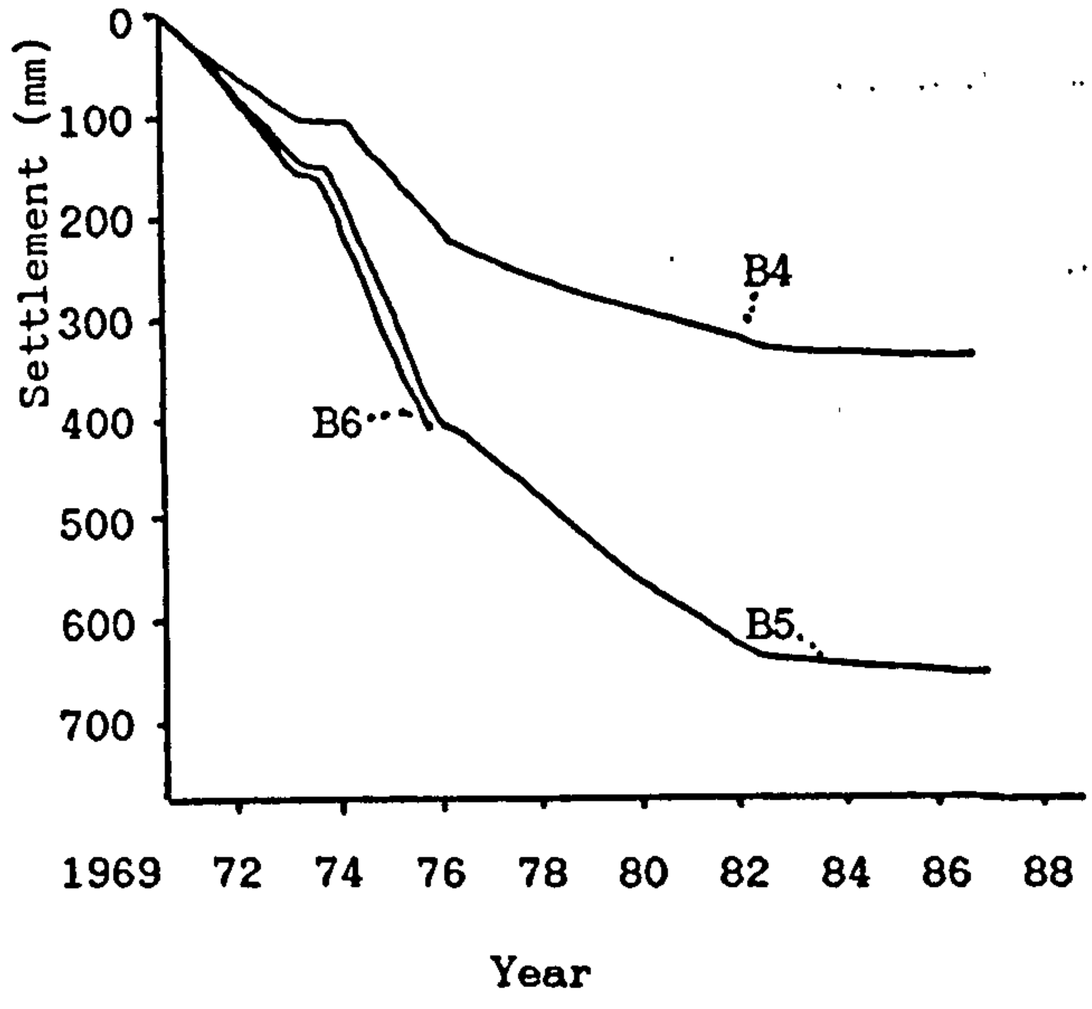


Fig. 13 Radar North, Traverse B settlement

Traverse B, Monitoring from 1969

Stn	Settlement mid1987 (mm)	Fill Depth (m)	Settlement %fill depth
B1	+19	0	-
B2	+17	0	-
B3	+17	0	-
B4	-336	35	0.96
B5	-644	55	1.17
B6*	-402	55	0.73

* defunct after 11.2.76

Table 3. Magnitudes of backfill settlement; Radar Site

5.3 Traverse C

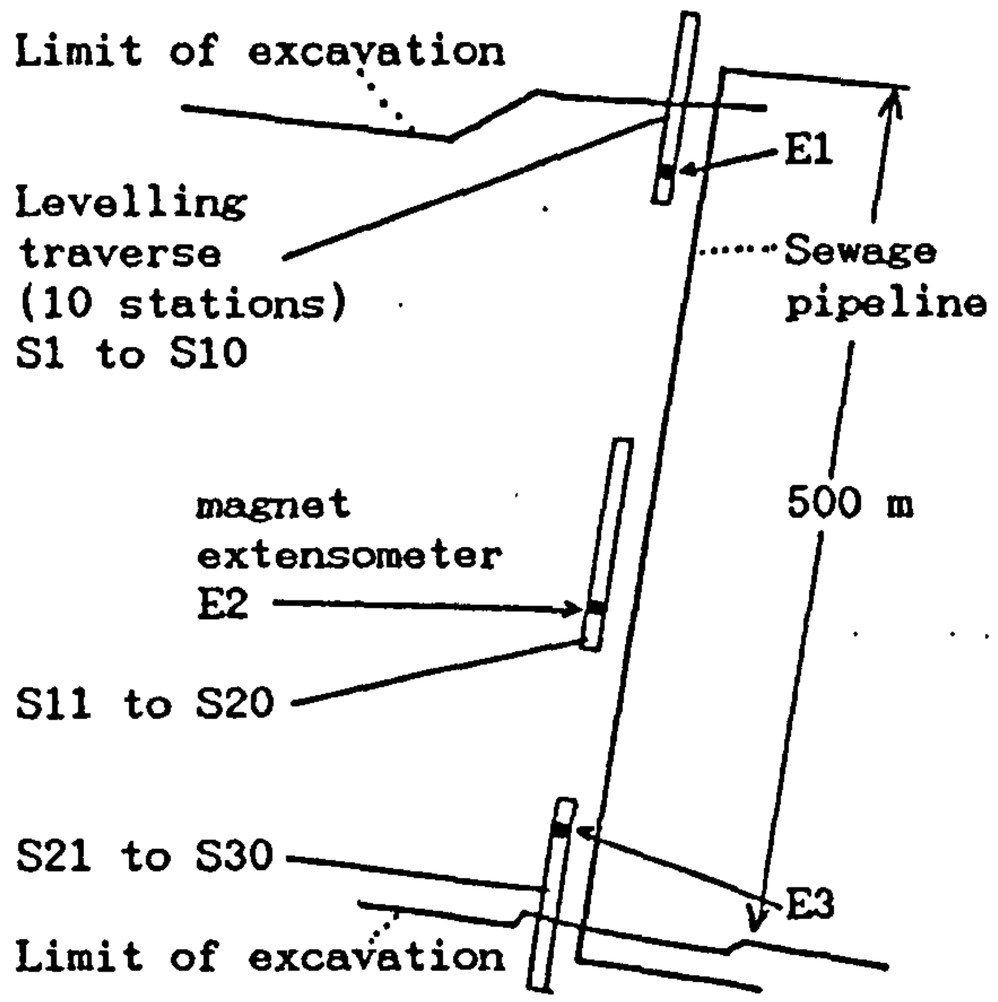
Eight levelling stations have been monitored from 1983 onwards. Small displacements have been observed over this period in the range +17 to -63 mm. With such a small range of displacements no trends were identified. It is presumed that a great deal of settlement had been completed prior to monitoring.

5.4 Summary

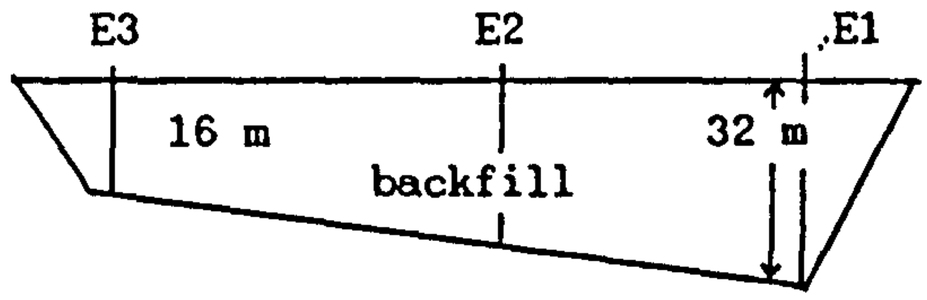
This case reinforces the concept of long term movements of significant magnitude. The role of the slowly recoverying groundwater table has been indicated as the main direct controlling factor for both traverses monitored. In this case however after the completion of water recovery, the settlement rate, (i.e creep settlement), has been practically nil.

6 CASE STUDY 4. SISTERS SITE

The project consisted of the monitoring of the settlement of a sewage pipe line constructed on backfill restored in 1974. The project commenced in 1982, and the scheme is presented in figure 14. Instrumentation consisted of three magnetic extensometers/ piezometers in the fill complemented by 3 sets of 10 surface levelling stations. Seven manhole covers also acted as surface levelling points. The design of the pipeline was such that some degree of settlement could be tolerated by the introduction of flexible joints between the individual pipe sections.



a) Plan of instrumentation



b) Section along line of sewer

Fig. 14 Sisters site instrumentation

6.1 Surface displacements

The time elapse between the date of backfilling and the commencement of monitoring was some 7.5 years. In the period of monitoring only small movements have been noted, either settlement or heaving movements. No distinct trends in settlement have been observed. The fill varies from 16 to 32 m deep. Table 4 details the total displacements recorded on all stations between 1982 and late 1988.

By inspection the fill can be seen to be fairly stable. All measurments are well within the tolerable design limits of the flexible sewer, and only one result, MHE, (77mm), can be said to have displaced with any significant magnitude.

6.2 Groundwater observations

The piezometers installed as part of the extensometer instruments have recorded fluctuating water levels within the fill over the monitoring period. On instrument E3 there is the indication that the fill has been saturated and dry in the upper 16 m. No evidence of collapse settlement has been observed at any time.

Extensometers, (Total settlement)

E1	-23	E2	+7	E3	+9

Manhole covers

MHE	-77	MHF	+5	MHG	+8
MHH	+5	MHJ	-5	MHK	0

(situated at equal spacings along sewer MHE close to E1)

Surface levelling stations

S1	+3	S2	+2	S3	-8	S4	-7
S5	-7	S6	-19	S7	-21	S8	-23
S9	-23	S10	+10	S11	+10	S12	+10
S13	+10	S14	+7	S15	+6	S16	+7
S17	+5	S18	+4	S19	+6	S20	+6
S21	+6	S22	+7	S24	+7	S25	+5
S26	+7	S27	+6	S28	+6	S29	-9
S30	0						

settlements in mm
results not available for S23

Table 4. Sisters Site, settlement results 1982-1988

6.3 Summary

The Sisters site illustrates that stability may be achieved in uncompacted mine backfills, and in this case a period of at most 7.5 years after backfilling provided a fill exhibiting very little in the way of movement. The delay in monitoring meant that it was inevitable that the early behaviour of the fill was lost. The case however does prove that uncompacted backfills can ultimately form a stable foundation for such structures.

7 CONCLUSIONS

The following conclusions can be drawn from this work.

a). The settlement rates of uncompacted backfills have been observed to be very variable in terms of both magnitude and rate.

b). Both the rates and magnitudes of displacement are affecting by a complex interaction of mining method, mine configuration, material properties and groundwater.

c). Long term backfill movements of over 10 years have been observed. These movements do not show, at this moment in time, any signs of retarding.

d). The siting of structures, even of the most simplistic nature, e.g. water channels, require detailed site investigation when siting on backfill materials regardless of age. The nature of the entire fill depth requires assessment, in order to establish material properties, voids and groundwater behaviour. The investigation should consider external influences, e.g. remote pumping and their likely effect on backfill stability.

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PAPER C13 GEOTECHNICAL STABILITY REPORT

Hughes, D.B. 1991.

The Geotechnical Stability Report.

Health and Safety Executive Conference on Geotechnical Codes of Practice for Opencast Coal Sites, University of Leicester.

THE GEOTECHNICAL STABILITY REPORT

D.B. Hughes, Regional Geotechnical Engineer, (Northern Region),
British Coal Opencast.

Health and Safety Executive Conference on Geotechnical Codes of Practice for Opencast Coal Sites, University of Leicester, 19th March 1991.

1. INTRODUCTION

Since the introduction of the Code of Practice for Excavated Slopes (Reference 1) into British Coal Opencast (B.C.O.) coaling contracts in January 1990, the B.C.O. geotechnical team for Northern Region has so far completed four Geotechnical Stability Reports (G.S.R.)s which attempt to comply with the Part 1 requirements of this Code. At least two more G.S.R.s are in the fairly advanced stages of preparation, and many more sites are at present at earlier stages of investigation and report compilation.

In addition, geotechnical site investigations which comply with the Part 1 requirements of the Code of Practice for Spoil Mounds (Reference 2) have been carried out in Northern Region for around thirty sites since the introduction of that Code in September 1982. The results of these investigations are normally summarised in Addendum Q Part 1 of the B.C.O. coaling contract.

This paper sets out to give a brief account of the geotechnical investigations which are carried out in order to meet the Part 1 requirements of both Codes of Practice, and to show the current format for the G.S.R. which has evolved in B.C.O. Northern Region.

2. CONTENTS OF THE GEOTECHNICAL STABILITY REPORT (G.S.R.)

2.1 PART 1 REQUIREMENTS OF THE CODE OF PRACTICE FOR EXCAVATED SLOPES

The information to be provided in the G.S.R. is set out in paragraphs 5, 6 and 7 of Part 1 of the Code of Practice for Excavated Slopes, and these are summarised (and much abbreviated) in Table 1. Paragraphs 5, 6 and 7(ii) & (iii) cover the requirements with regard to factual data, whereas the remainder of paragraph 7 is mainly concerned with assessments of the effects on stability of the various physical and geological conditions at the proposed opencast site. Paragraph 6 also relates to stand-off distances against features and services within and/or adjacent to the site. It can be seen from Table 1 that the main emphasis in Part 1 of this Code of Practice is the provision of factual data, and that the information is collected under the following five headings:-

(i) SURFACE AND SHALLOW FEATURES
(ii) SUPERFICIAL DEPOSITS AND ROCKHEAD
(iii) BEDROCK
(iv) PREVIOUS WORKINGS
(v) GROUNDWATER

The work done under (i) and (iv) above tends to comprise the desk study or preliminary phase of the site investigation, whereas (ii), (iii) and (v) all require direct forms of ground exploration (boreholes, trial pits etc.) and together form the main geotechnical site investigation. A brief account of the geological conditions as experienced at U.K. opencast coal sites, plus details of methods of carrying out ground investigations was given at the DRILLEX '87 Conference (Reference 3).

B.C.O. has available two ground investigation term contracts, one intended mainly for coal prospecting, and the other for geotechnical work including insitu and laboratory testing. Prospecting work usually entails a far greater density of boreholes than geotechnical work, and hence the prospecting fieldwork phase normally covers a rather longer time span. Much of the information relating to geological structure and strata succession, which is gained from normal prospecting activities, forms the basic framework onto which the geotechnical data and parameters are added. Therefore, geotechnical field and laboratory investigations are mostly carried out in the latter stages of prospecting work, and as an extension thereof. In the majority of cases geotechnical bedrock drilling is carried out under the prospecting contract, although for some complex situations the geotechnical contract is preferred. Boring, sampling and testing in the superficial deposits is nearly always carried out under the geotechnical contract.

Appendix 1 (of this paper) lists the sources of information, records, features, methods of investigation and parameters, as appropriate to each of the five headings given above. These are not intended to be complete lists, and more comprehensive information may be found in the 'Handbook on the Hydrogeology and Stability of Excavated Slopes in Quarries', (Reference 4). Also, not all the sources of information listed are available for every site.

2.2 B.C.O. NOTES FOR GUIDANCE (Reference 5)

B.C.O. Headquarters has produced 'Notes for Guidance' on the Code of Practice for Excavated Slopes for the Regional Geotechnical Engineers and Project Team Leaders to use. These 'Notes for Guidance' follow the same format as the Code but contain more information, including references to the division of the work between geotechnical staff and the project teams. General guidance is given on methods of investigation, information which should be provided in the G.S.R., and the required extent of the investigations in relation to the proposed site boundary and pavement seam.

2.3 THE FORMAT FOR THE G.S.R.

Table 2 shows the format for the G.S.R. as currently used by B.C.O. Northern Region. The 'Preliminaries' section includes a list of all relevant Codes of Practice, Contract Clauses, Appendices (internal and external), and all other reference documents. This is followed by 'Field and Laboratory Investigations' which contains accounts of the methods of investigation used and summaries of the factual data and parameters obtained, generally under the main headings given in Table 1.

The engineering aspects and feasibility of working a proposed opencast site are continually reviewed by B.C.O. project teams and geotechnical staff from the early prospecting stages, through the planning liaison and application stages, until the final contract documents are produced. Hence, the 'Assessment of Factors Affecting Stability' section discusses the anticipated geotechnical problems and their possible solutions as revealed during this long investigation and consultation process. As well as excavated slopes, other construction works are also discussed in this section e.g. lagoons, spoil mounds, diversions of services etc. Any agreements made by B.C.O. with third parties regarding safety stand-off distances or batter profiles etc., are also described (as well as being detailed in the contract specification clauses).

TABLE 1 CODE OF PRACTICE FOR EXCAVATED SLOPES - FACTUAL DATA (Paras. 5 & 6) AND ASSESSMENTS (Para. 7) - PART 1 REQUIREMENTS

SURFACE AND SHALLOW FEATURES	SUPERFICIAL DEPOSITS AND ROCKHEAD	BEDROCK	PREVIOUS WORKINGS	GROUNDWATER
Factual Data				
5(iii) existing surface instabilities	5(i) thickness, nature and properties	5(v) lithological and engineering descriptions of all strata from rockhead to below pavement	5(x) underground workings within, below and adjacent to the site	5(ii) groundwater in superficial deposits
5(xi)* surface excavations within and adjacent to the site	5(iv) rockhead dips		5(xi) *surface excavations within and adjacent to the site	5(ix) groundwater in bedrock
6(i) all services and features	7(ii)*measured or estimated shear strengths	5(vi) bedding dips	6(iii)*active and abandoned mineral workings (inc. surface excavations, tips, lagoons, shafts, adits, working levels, buried & backfilled excavations)	7(iii) groundwater levels and pressures
6(ii) abandoned services				
6(iii)*active and abandoned mineral workings (inc. surface excavations, tips, lagoons, shafts, adits, working levels, buried & backfilled excavations)		5(vii) low shear strength horizons		
		5(viii) faulting and structural features		
6(iv) natural drainage features, springs seepages, watercourses		7(ii)* measured or estimated shear strengths		

Assessments

7(i) description of each failure mode considered

7(v) instabilities in superficial deposits	7(iv) potential interaction of working faces, geological features, directions of advance	7(vii) effects of active and abandoned workings below, within or adjacent to the site	7(viii) effects of groundwater on excavated and backfilled slopes
	7(vi) in-pit spoiling		

TABLE 2

GEOTECHNICAL STABILITY REPORT - FULL CONTENTS AND APPENDICES

CONTENTS

1. PRELIMINARIES

   Location and Size of Site,
   Codes of Practice - Contract Clause Nos.
   Reference Documents and Appendices

2. FIELD AND LABORATORY INVESTIGATIONS
   (Factual data and Summaries)

   Geological and Prospecting Summaries
   Records from nearby sites
   Surface and Shallow Features
   Superficial Deposits and Rockhead
   Bedrock
   Previous Workings(Surface & Underground)
   Surface Water and Groundwater

3. ASSESSMENT OF FACTORS
   AFFECTING STABILITY

   Method(s) of Working
   Surface and Shallow Features
   Construction of Spoil Mounds
   Construction of Lagoons
   Construction of Ancillary Works
   (highway or service diversions etc.)
   Excavations in Superficial Deposits
   Excavations in Bedrock
   Previous Workings
   Surface Water and Groundwater

4. OTHERS
   e.g. Contaminated Land

   SIGNATURE OF COMPETENT PERSON

   APPENDICES

INTERNAL APPENDICES

Descriptions and diagrams of types of slope failure in bedrock and superficial deposits

Lists of all archive and other documents referred to for information on:-
e.g. Shaft and adit locations etc.
Old mining and quarrying records
Underground and overhead services
Industrial archaeology
Contaminated land etc, etc.

Rainfall records

Groundwater level records (piezometer and standpipe details)

Geotechnical bedrock (cored) borehole and bedrock testing results

Bedrock discontinuity measurements for outcrops and exposures, or from adjacent working sites

Comparison with Part 1 requirements of the Excavations Code of Practice

Geotechnical Plans and Sections
e.g. Geotechnical borehole plan
Surface and shallow features plan
Geotechnical composite mining records plan(s) and sections

EXTERNAL APPENDICES

Geotechnical Site Investigation Report(s) for Superficial Deposits(may be several volumes) including:-
Borehole log sheets.
In-situ and laboratory test results.
Schematic borehole sections.

(For Bedrock the details of the)
(Geotechnical investigation part of the)
(prospecting programme are given with the)
(Geological documents listed below and in)
(the appropriate Internal Appendices.)
(However, separate geotechnical bedrock)
(investigations are sometimes carried out.)

Final Geological Report
Borehole Schedule (prospecting)
Geological Plans and Sections:-
e.g. Geological key plan
Structure plans and sections
Superficial deposits thickness plan
Rockhead level plan
Seam old workings plan(s)
Fully cored borehole details

Contract Site Plan
Contract Specification
Schedule of Estimated Quantities

Other relevant reports:-
e.g. Deep Mines Tip site investigation reports
Mineral Valuer's Report
Subsidence reports
Sand and Gravel reports

Finally, the 'Appendices' (internal and external) should together contain "all factual data" as required by paragraph 7 of Part 1 of the Code of Practice for Excavated Slopes. Appendix 2 (of this paper) shows a comparison of the information provided with the requirements of Part 1 of the Code.

3. SOME NEW DEVELOPMENTS

The following are some geotechnical developments on which work has recently been carried out or has just begun. It is not suggested that all of these developments have arisen solely from the introduction of the Code of Practice for Excavated Slopes, but certainly some increase in priority and momentum has resulted.

3.1 GEOTECHNICAL CORE LOGGING

A 'Field Manual of Geotechnical Core Logging' (Reference 6) has recently been completed, the purpose of which is to provide a format for the consistent geotechnical description of cores logged by B.C.O. geologists and geotechnical engineers. It is intended that this manual should become a 'Quality Assurance' document. Also, a companion volume, the 'Geotechnical Investigation Handbook' is currently in preparation.

3.2 GEOTECHNICAL SITE INVESTIGATION CONTRACT

As mentioned earlier, B.C.O. has available two ground investigation term contracts, one for geotechnical investigations and one for prospecting work. Both contracts are revised from time to time and the geotechnical investigations contract is currently being extensively revised. In the new version it is intended to include the investigation of contaminated land, and to introduce some form of 'Quality Assurance'.

3.3 OLD WORKINGS RECORDS AND SUBSIDENCE

It is the intention to produce composite (geotechnical) old mine workings plans for each proposed opencast site, and wherever possible an assessment of the subsidence effects and ground strains. There are many new developments taking place in association with the various Mining Records Offices and especially with British Coal Headquarters Technical Department at Bretby. These include microfilming, digitising and rationalising of Abandoned Mine Plans and other records, and the use of Autocad to produce the final composite plans.

3.4 BEDROCK - WEAK HORIZONS RESEARCH

In October 1990 a research project into the shear strength of weak horizons in Coal Measures bedrock was started by B.C.O. Northern Region and Newcastle University. The main aims of this project are to obtain suitable test specimens of weak horizon materials (e.g. fault gouge, weak seatearths, clay bands, "mylonites", and intra-formational shear zones etc.) both from prospecting and geotechnical drilling investigations, and from exposures at operating opencast coal sites. Hopefully, improved methods of sampling and testing of these materials will be developed, and a database of shear strength results will be created for the North of England. A similar project has also started at Leeds University on behalf of B.C.O. South Wales Region, and much work has previously been done in the Midlands by Nottingham University.

4. SOME PERSONAL VIEWS

4.1 RESPONSIBILITY OF THE COMPETENT PERSON FOR PART 1

It appears that total responsibility for the content and accuracy of the G.S.R., and the Contract Addendum Q Part 1, rests with the individual Regional Geotechnical Engineer (or an individual from a firm of specialist geotechnical consultants - if this work is contracted out by B.C.O.) who signs these documents as the Competent Person for Part 1 of either or both of the Codes of Practice (see INTRODUCTION). That one named person takes sole responsibility seems to be a feature of Health and Safety Executive inspired Codes of Practice.

It is the author's view that this cannot be applied to all the information as set out in Tables 1 and 2, particularly many of the documents listed as external appendices, where the Regional Geotechnical Engineer (or other Competent Person) has had little or no involvement in their preparation or in the gathering of such information. For example, the Final Geological Report, Prospecting Borehole Schedule, and all the supporting geological plans and sections are prepared by the B.C.O. project teams and not by the geotechnical team. Documents listed under 'Other Relevant Reports' are also offered in good faith.

Geotechnical ground investigations are carried out by specialist contractors, and whilst a token amount of fieldwork supervision is undertaken, most of the information provided (especially laboratory testing) is taken on trust. Hence the proposed introduction of 'Quality Assurance' requirements into the geotechnical investigations term contract (see previous section 3.2)

Most of the documents referred to in the previous two paragraphs carry the names or signatures of those persons who are responsible for their preparation and content. In the G.S.R. a caveat referring to these other documents usually precedes the name and signature of the Competent Person.

4.2 FEEDBACK AND LIAISON BETWEEN PARTS 1 AND 2 COMPETENT PERSONS

When the Competent Person for Part 1 (usually the B.C.O. Regional Geotechnical Engineer) is preparing the G.S.R. he/she has no idea who the Competent Person for Part 2 is going to be or what his/her preferred analysis methods are, and consequently which are the required geotechnical parameters (e.g. will the spoil mounds be analysed by total stress or effective stress methods?). Therefore, the Competent Person for Part 1 must always adopt a "catch-all" approach to the site investigations (i.e. a high density of boreholes and samples, plus a wide variety of laboratory tests).

The Competent Person for Part 2 can seek any additional site investigation information which he/she may require (paragraph 8 of the Code of Practice for Excavated Slopes), but there are likely to be severe time constraints on carrying out any additional work of this type, i.e between the Contract being awarded and excavation works commencing. To ensure that future G.S.R.s are always being improved and contain the relevant information, it is suggested that the various Competent Persons should occasionally discuss these matters.

5. ACKNOWLEDGEMENTS

This paper was written at the request of the Health and Safety Executive, and with the encouragement of British Coal Opencast. The constructive criticisms and helpful comments given by colleagues from B.C.O. Northern Region are gratefully acknowledged. However, the views expressed herein are entirely those of the author.

6. REFERENCES

1. Code of Practice; 'The Stability of Excavated Slopes at Opencast Coal Sites'; Federation of Civil Engineering Contractors, London, October 1989.

2. Code of Practice; 'The Siting and Construction of Temporary Spoil Mounds at Opencast Coal Sites'; Federation of Civil Engineering Contractors, London, September 1982.

3. Subsurface Investigations by Drilling and Boring for Opencast Coal Mining Projects in the United Kingdom; D.B. Hughes and D.A. Blythe; DRILLEX '87 Conference; Institution of Mining and Metallurgy, London, 1987.

4. Handbook on the Hydrogeology and Stability of Excavated Slopes in Quarries - for the Department of the Environment; (prepared by Geoffrey Walton Practice); HMSO, 1988.

5. Notes for Guidance for the Code of Practice 'The Stability of Excavated Slops at Opencast Coal Sites'; British Coal Opencast; December 1989 - (for B.C.O. internal use).

6. Field Manual of Geotechnical Core Logging - British Coal Opencast; (prepared by Wimpey Environmental Ltd.) October 1990.

APPENDIX 1 - INVESTIGATIONS AND FACTUAL DATA FOR THE G.S.R.

1. SURFACE AND SHALLOW FEATURES

Features

(i) Surface contours, proposed site boundaries and limits of excavation.
(ii) Current and abandoned services (underground and overhead).
(iii) Buildings, ruins, foundations, evidence of former structures and earthworks.
(iv) Railways, roads, tracks, paths, culverts, bridges, tunnels etc.
(v) Watercourses (with flow directions) including rivers, streams, ditches, canals etc.
(vi) Lakes, ponds, reservoirs, lagoons.
(vii) Springs, water issues, wells, waterlogged/marshy ground, sink/swallow holes etc.
(viii) Crags, outcrops, escarpments, landslips, hummocky ground, steep slopes, depressions and hollows, ravines and gulleys.
(ix) Quarries (abandoned or operational), spoil heaps, shafts, adits, pit-fallen and subsided ground (bell-pits, crownholes etc.), backfilled and contaminated areas.
(x) Vegetation and cultivation details (e.g. woodland, scrub, heather, bracken, arable, pasture, waste or derelict land, rig and furrow systems etc.).

Potential Sources of Information

(i) Topographical surveys - (aerial photogrammetry and/or land surveying).
(ii) Walk-over surveys and mapping (and conversations with long standing local residents).
(iii) Aerial photographs - stereoscopic studies.
(iv) County Records Office - all current and previous O.S. editions, estates and tithe plans.
(v) British Geological Survey - sheets and memoirs.
(vi) Mining Records Office - Abandoned Mine Plans, shaft records, and colliery spoil tip records etc.
(vii) Statutory Undertakers - correspondence and records.
(viii) Mineral Valuer.
(ix) Agricultural drainage plans - A.D.A.S. soil and drainage surveys.
(x) Other public and private records - (e.g. trade directories).

2. SUPERFICIAL DEPOSITS AND ROCKHEAD

Methods of Investigation

(i)	Trial pits	- samples for laboratory testing, (occasional insitu tests).
(ii)	Cable percussion boring	- insitu tests and samples for laboratory tests.
(iii)	Openhole rotary drilling	- approx. succession and rockhead.
(iv)	Geophysical traverses (occasional)	- approx. succession and rockhead.
(v)	Hand-augering/probing (occasional)	- in soft alluvium and peat etc.

Information and Parameters/Test Results

(i) Superficial deposits succession and engineering descriptions.
(ii) Superficial deposits thickness and rockhead levels/gradients.
(iii) Relative density in granular materials (SPT, CPT).
(iv) Shear strength parameters (total, effective, peak, residual).
(v) Classification (Atterberg Limits, particle size distribution).
(vi) Moisture content, compaction, CBR, consolidation.
(vii) Chemical tests (SO_3 and pH usually + additional tests where contamination is suspected).

3. BEDROCK

Methods of Investigation

(i) Prospecting) - Rotary openhole air-flush drilling.
) - Slim-line (geophysical) logging.
and) - Rotary cored air-flush drilling
) (both 'seam' (+ roof & floor) cores)
(ii) Geotechnical) (and full succession 'strata' cores).
- (water, mud and foam flush occasionally used).
- Engineering core logging.
- Measurements at nearby exposures in quarries or opencast sites.
- Direct shear testing (especially for weak horizons).
- Uniaxial compression and/or point load index testing (occasional).

Information and Parameters/Test Results

(i) Geological structure and bedrock succession.
(ii) Engineering descriptions and logging indices, (e.g. core recoveries, R.Q.D., fracture logs etc.).
(iii) Discontinuity spacing and orientation - mainly bedding and faulting, (occasionally jointing from nearby exposures in quarries or opencast sites, or possibly inclined boreholes or sonic logging (Televiewer)).
(iv) Shear strength parameters for weak horizons (weak seatearths, fault gouge, "clay mylonites", clay bands, intra-formational shear zones etc.).
(v) Uniaxial compressive strengths (occasional).

4. PREVIOUS WORKINGS

Information Required

(i) Areas of pillar and stall, goaf, and longwall workings.
(ii) Locations of shafts, staples, drifts, adits, bell-pits and crownholes etc.
(iii) Faults recorded in underground workings and previous opencast sites.
(iv) Areas of flooded workings etc. (see GROUNDWATER).
(v) Locations and magnitudes of subsidence and ground strains.

Potential Sources of Information

Generally as for SURFACE AND SHALLOW FEATURES, plus Department of the Environment "Review of Mining Instability in Great Britain", (prepared by Ove Arup and Partners, Consultants) 1990.

5. GROUNDWATER

Methods of Investigation, and Information Required

(i) Piezometers/standpipes in boreholes - for groundwater levels/pressures (in superficials and bedrock).
(ii) Water sampling from boreholes and surface - for water quality.
(iii) Insitu permeability testing in boreholes - (occasional).
(iv) Areas of flooded old workings and interconnections.
(v) Details of present or former minewater abstraction.

Additional Information:-

(i) Rainfall records - (Met. Office, plus other private weather stations).
(ii) Surface watercourse flow monitoring records.

APPENDIX 2 EXAMPLE FROM PLENMELLER G.S.R.

Comparison of information provided with paras. 5, 6 & 7 of Part 1 of the Code of Practice for Excavated Slopes (G.S.R. - Geotechnical Stability Report) (F.G.R. - Final Geological Report)

5. The following information shall be collected:-	Information presented in:-
i. The thickness, nature and properties of any superficial deposits overlying rock head	FGR Section 3.2, Geological Supplements - Peat and Superficial Deposits, Geotechnical(Superficials) Sections, GSR Section 2.4, Ground Investigation Report for Superficial Deposits
ii. Groundwater levels present within the superficial deposits	GSR Section 2.8.3 (Table 6, Figures 2 to 4) Appendix 5 - Groundwater Records Groundwater Investigation Report for Superficial Deposits - borehole logs
iii. Any existing surface instability affecting superficial and/or bedrock materials	Surface and Shallow Features Plan GSR Section 2.2
iv. The directions and amounts of rock head dips	FGR Section 3.2.4, Geological Supplement - Rockhead Contour Plan, GSR Section 2.5
v. Lithological and engineering description of strata including partings from rock head to below basal seam to be worked	FGR Section 3.3, GSR Sections 2.6, Geological Cross Sections, Ground Investigation Report for Bedrock (Volume 2 - Borehole logs) Borehole Schedule (Prospecting)
vi. The directions and degree of bedding plane dips	FGR Section 3.3.1, Geological Plan, Geological Structure Plans, Geological Cross Sections, GSR Section 2.6.1
vii. The extent and properties of low shear strength horizons in seatearths or clay bands within mudstone or elsewhere by reference to lithological descriptions, sampling and testing and where possible from experience on neighbouring working sites	GSR Sections 2.6.2 and 2.6.3 (Tables 2,3,4 & 5) Ground Investigation Report for Bedrock (Volume 1 - test results; Volume 2 - geotechnical borehole logs)
viii. The presence and details of faults and other structural features	FGR Section 3.3, Geological Plan, Geological Structure Plans, Geological Cross Sections GSR Section 2.6.1
ix. Groundwater levels and pressures in the strata below rock head	FGR Sections 5.2 and 5.3, GSR Section 2.8.4, (Table 7 and Figures 5 to 7) Appendix 5 - Groundwater Records
x. Information in respect of all underground mineral workings within, below and adjacent to the site	FGR Sections 4.1 to 4.4, GSR Sections 2.7.2 & 2.7.3, Geological Supplements - Old Workings, Geotechnical Composite Old Workings Plan. Appendix 4 - Records of Old Drifts and Shafts
xi. Information in respect of the extent of all surface excavations within and adjacent to the site	FSR Sections 4.5 and 4.6, GSR Sections 2.2 & 2.7.1
xii. Any other relevant geotechnical or hydrogeological data	Appendix 6 - Rainfall Records Consultants Report (BSM 1)

6. The following information should be collected in relation to stand off distances against features/services within, and/or adjacent to the site	Information presented in:-
i. Location of all services and features. Features and services which may need protection include all buildings and contiguous land, railways, roads and footpaths, pipelines for water, sewerage, gas, oil, etc., power lines (underground and overhead), canals, rivers and reservoirs, areas of established amenity or scientific interest and and areas of public access	Site Plan, Surface and Shallow Features Plan
ii. Location of all abandoned services that might jeopardise slope stability (e.g. abandoned sewers, drains etc.)	Site Plan Surface and Shallow Features Plan
iii. Location of both active and abandoned mineral workings, surface excavations, tips and lagoons, including the position of shafts, adits and working levels, benches and faces as well as the limit of previous buried, backfilled excavations	FGR Section 4, GSR Section 2.7, Geological Plan – Old Workings Supplement, Geotechnical Composite Old Workings Plan, Surface and Shallow Features Plan, Appendix 4 – Records of Old Drifts and Shafts
iv. Location of all natural drainage features including springs, seepages and watercourses	Site Plan, Surface and Shallow Features Plan Consultants Report (B.S.M.I.)
A note should be made of any data which it has not been possible to obtain	All known and available records have been consulted, (See Appendix 3 – Documents referred to for Part 1, Para. 6 of the Code of Practice For Excavated Slopes). Should any further records which are considered relevant be discovered subsequent to the Contract being awarded, then these will be forwarded to the successful Tenderer, (as Quarry Owner)

7. The information revealed from the procedures above shall be collated by the Competent Person to form a Geotechnical Stabilility Report which will provide a basis for the design of slopes and determination of stand off distances. The report will contain all factual data and detail the geotechnical aspects of the site including identification of likely modes of ground failure	
The Geotechnical Stability Report shall include:-	Information presented in:
i. A description of each failure mode considered:	GSR Section 3 Appendices 1 and 2
ii. Details of measured or estimated shear strengths;	GSR Section 2 (Tables 1,2,4,5 and Figure 1) Ground Investigation Report - Superficials and Bedrock Contract Dwg. Nos. 115 A to F
iii. Details of ground water levels and pressures;	GSR Section 2.8 (Tables 6 & 7, Figures 2 to 7) Appendix 5 - Groundwater Records
iv. An assessment of the potential interaction of working faces and known geological discontinuities for various directions of advance excavation;	GSR Section 3.1, 3.6, 3.8
v. An assessment of potential instability arising from any superficial deposits;	GSR Sections 3.2, 3.3, 3.4, 3.5
vi. An assessment of potential instability in backfilled areas arising from interaction of dip and/or nature of backfill pavement;	GSR Section 3.8
vii. An assessment of the potential effect of active or abandoned mineral workings below, within, or the site	GSR Section 3.7
viii. An assessment of the potential effects of ground water on excavated slopes and backfilled slopes	GSR Section 3.9

PAPER C14 SPOIL MOUNDS 1

Blythe, D.A., Hughes, D.B. and Clarke, B.G. 1993.

Design and construction of spoil mounds.

Proceedings of the International Conference on Engineered Fills, Newcastle upon Tyne (eds. C.J.F.P. Jones, B.G. Clarke, A.I.B. Moffat), Thomas Telford Ltd., 516-528.

41. Design and construction of spoil mounds

D. B. BLYTHE and D. B. HUGHES, British Coal Opencast, UK and
B. G. CLARKE, University of Newcastle upon Tyne, UK

SYNOPSIS. British Coal Opencast have undertaken a study of the performance of a subsoil mound to facilitate the design of future mounds. Subsoil mounds are constructed rapidly to guidelines which cover the geometry of the mound. In this study the mound was constructed in layers and observations were made of the pore pressure response in the foundation soils and the settlement of the mound. Samples of the foundation soils and subsoil mound have been taken and tested. These data have been used to establish the reasons for the failure of an earlier subsoil mound and to produce preliminary design charts for future development.

INTRODUCTION

1. The extraction of coal by opencast mining is an established economical technique which involves moving a substantial amount of material in order to win the coal. This material can include superficial deposits such as topsoil and subsoil, plus glacial deposits and Coal Measures strata which are known as overburden. In Northumberland, the extraction of one tonne of coal may involve the removal and storage of up to twenty cubic metres of overburden. Individual opencast mining projects may yield from half a million to twelve million tonnes of recoverable coal.

2. It is a requirement on the operator of a mine to restore the site so that other activities, for example agricultural, can take place. Therefore, the topsoil, subsoil and overburden have to be stored separately during the working life of the mine and then used to restore the land surface.

3. Each site will have, in order of increasing size, topsoil, subsoil and overburden mounds. These are constructed rapidly so that coal production can be achieved as quickly as possible. They remain in place throughout the working life of the mine which, in the Northumberland coalfield, can be between two and fifteen years.

4. Requirements for the ground investigation, design and construction of the mounds are described in a Code of Practice (ref. 1). Since these mounds are usually near the site perimeter and/or adjacent to excavated faces it is necessary to undertake stability analyses to ensure safety of services, adjacent properties and excavations. These analyses are usually based on total stress parameters using data from site investigations undertaken at the time of exploratory work for the mine. Where failures have occurred, it has been usually due to failure in the foundation soils.

5. As part of an on going research and development study following the failure of a subsoil mound, the Northern Region of British Coal Opencast undertook to monitor the performance of a more recently formed subsoil mound in order to determine the changes in the foundation soils and subsoil materials during its construction. This paper introduces this study and gives recommendations for the design of future mounds constructed of glacial clay subsoils founded on glacial deposits.

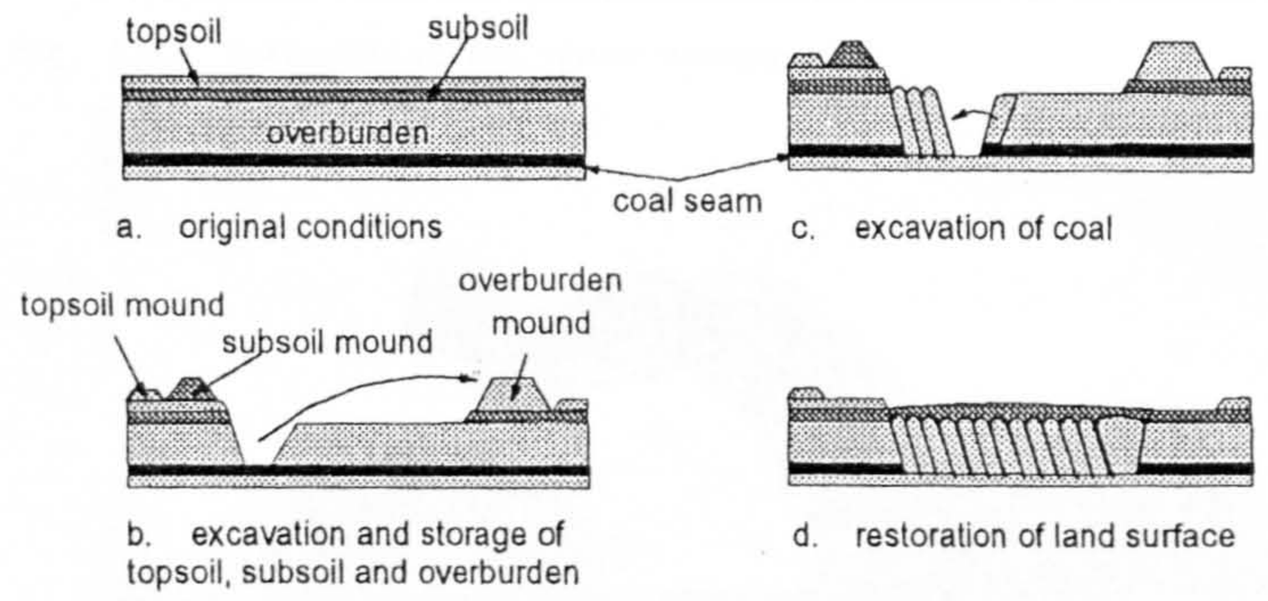

Fig. 1. The sequence of working for opencast coal mining

SEQUENCE OF OPERATIONS OF OPENCAST COAL MINING

6. The operations at an opencast coal site involve excavating through superficial deposits and Coal Measures strata in order to win coal from one or more seams. The simplified sequence of operations is as shown in Fig. 1. Firstly, topsoil and subsoil are excavated mainly by motor scrapers and transported for storage in mounds and dumps near to the site perimeter (1b). This is followed by excavation of the overburden to the coal seam(s) in the initial cut by face shovel or backactor. This overburden is transported by dump truck to the main overburden mound for storage. Successive cuts are then excavated by dragline and/or face shovel and/or backactor with spoils going into the previous cuts (1c). The final void is backfilled using the spoil from the overburden mound (i.e. from the initial cut), and the subsoil and topsoil are replaced (1d). Restoration to agriculture is the most usual end result, but other uses such as recreation, industrial

Engineered fills. Thomas Telford, London, 1993

development or forestry are also common. Fig. 1 shows that there is usually a net volume increase in the backfill materials, (bulkage), and this is accommodated in the final restoration contours.

METHOD OF CONSTRUCTION OF SUBSOIL MOUNDS

7. Typically subsoil mounds are ten to fifteen metres high, have base widths of 50 to 150m, and side slope gradients normally between 1 in 1.5 and 1 in 2. Subsoil mound lengths can be several hundreds of metres. Thus larger subsoil mounds can include up to one million cubic metres of material.

8. Generally, in the Northumberland coalfield, the subsoils consist of glacial materials which are predominantly till. This till, described by Robertson et al (1993), is generally stiff sandy silty clay with gravel, the stone content increasing with depth. The foundation soils of a storage mound usually comprise the same glacial materials. The till is weathered near the surface and contains lenses of laminated clays, sands and gravels. After the topsoil has been stripped the top one to two metres of till is generally used to construct a subsoil mound. It therefore mainly consists of weathered material, though other Units of till may be present.

9. Subsoil materials are normally excavated by motor-scraper and transported to the mounding area where they are deposited in loose layers of 0.5 to 1m thickness. Occasionally, the subsoil may be excavated by face shovel or backactor loading onto dump trucks. Some compaction takes place due to the successive passage of the motor scrapers or dump trucks over the previously deposited subsoil. However, compaction is not considered desirable since increased porosity improves drainage and preserves the soil structure for agricultural restoration. The subsoil is deposited in layers though not necessarily in sequence. For example, one end of a mound may be constructed first. The final shape of a mound is achieved using a dozer vehicle with a blade.

THE CONSTRUCTION OF THE COLLIERSDEAN SUBSOIL MOUND

10. During this study it was decided to monitor the construction of a mound and the effect that this had upon the foundation soils in order to produce parameters for design. The Contractor, RJB Mining Ltd, was required to construct the mound in sequence in a controlled manner, that is completing each layer before placing the next.

11. Fig. 2 shows the two methods of construction using motor-scrapers (Caterpillar 631) and dump trucks (Caterpillar 777). Table 1 gives details of the earth moving vehicles. In addition to these vehicles a Komatsu D155 and/or a Caterpillar D8 dozer was used to spread end-dumped subsoil, assist the unloading or discharging of motor scrapers and form the slopes of the mound.

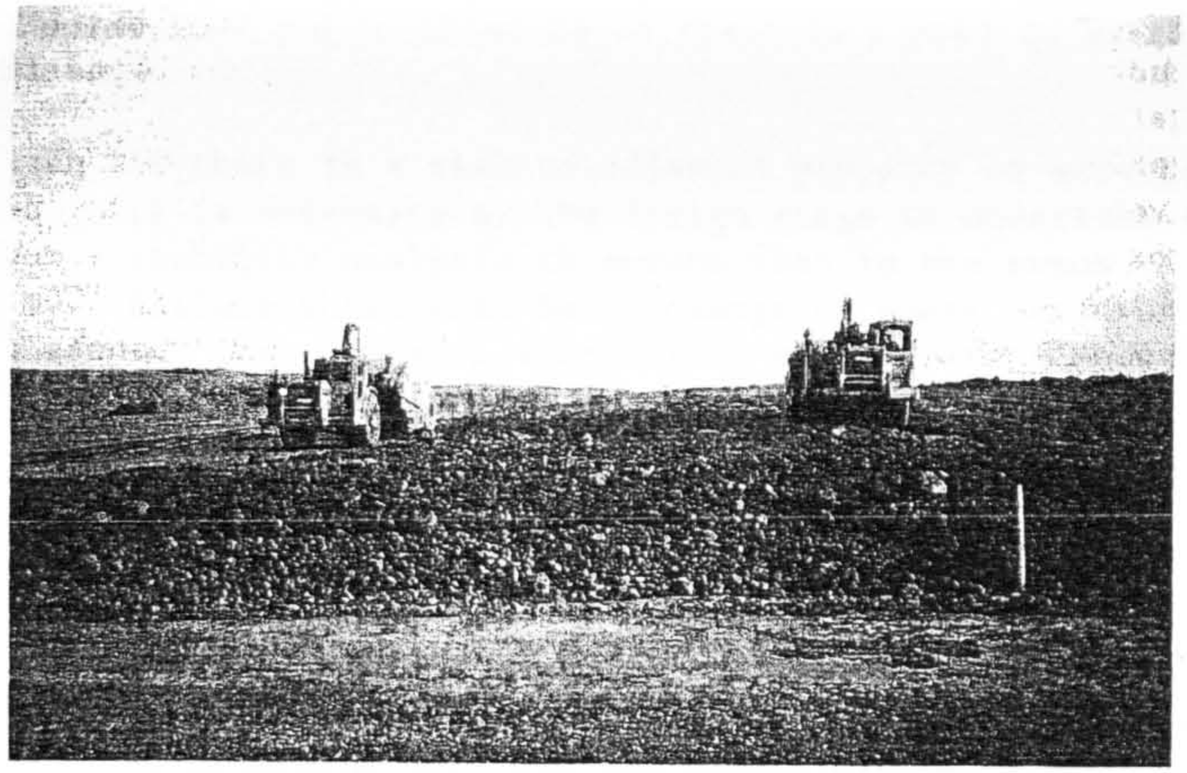

Fig. 2(a). Caterpillar 631 motor scraper

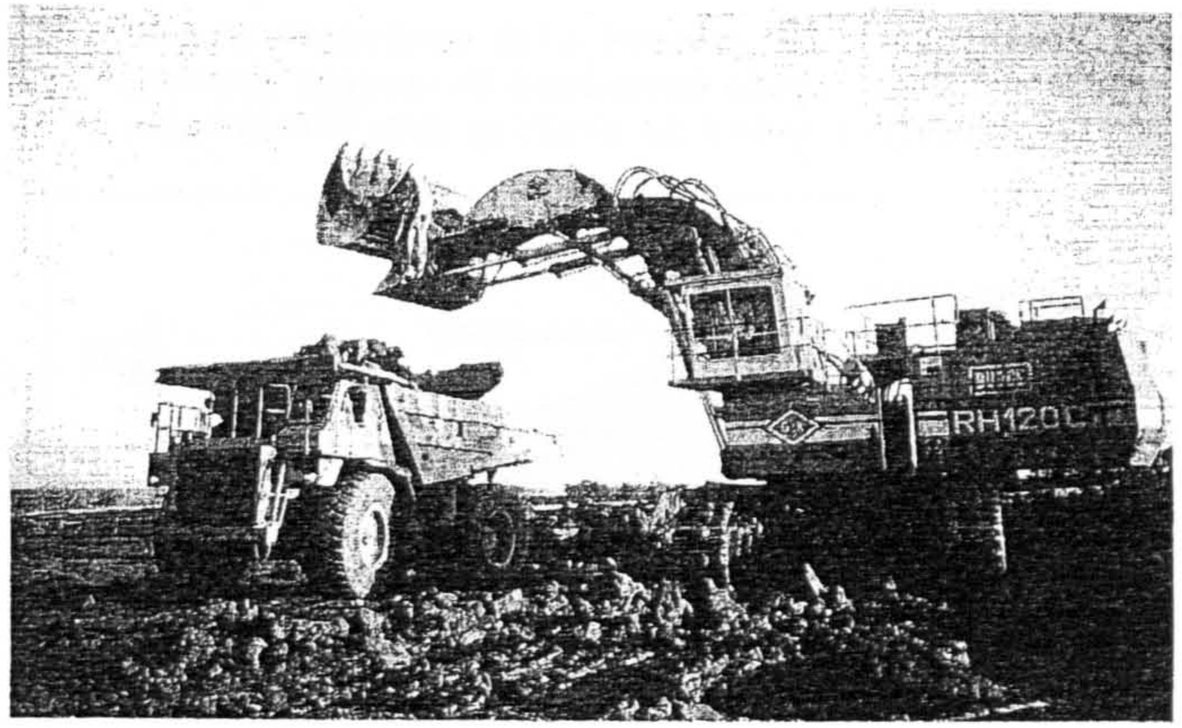

Fig. 2(b). Caterpillar 777 dump truck

Table 1 Earth moving equipment involved in the construction of Colliersdean subsoil mound

Vehicle	Unladen Weight tonnes	Laden Weight tonnes	Ground Contact Pressure/axle kN/m^2
Cat 631	39.24	73.26	376
Cat 777	55.6	132.7	386

631 - Caterpillar 631D motor scraper
777 - Caterpillar 777 dump truck

12. Table 2 gives a summary of the construction which includes, for each day, the number and type of vehicles in operation and the source of the material. It took approximately 37 working days (a day was for the most part a twelve hour shift) to construct the mound over a 50 day period. Approximately 370,000m^3 of subsoil was moved to form a mound 12m high with a base width of 140m, a base length of 230m and a side slope of 1 in 2. This is in excess of 0.60M tonnes of subsoil or about 20000 tonnes per day on average which is equivalent to 260 movements of a Cat 777 per day.

Table 2 The construction of Colliersdean subsoil mound

Date	Vehicle Type	No of Vehicles	Source of Subsoil	Height above gl
4/9/91 - 7/9/91	631	7 or 8	stripping topsoil	
9/9/91 - 14/9/91	631	7 or 8	Boxcut A	5
16/9/91 - 18/9/91	631 777	5, 7 or 8 2 or 3	Boxcut B	5
19/9/91 - 20/9/91	631 777	6 2 or 3	dragline corridor Boxcut B	7.5
23/9/91	631	6 or 8	dragline refurbishment	7.5
24/9/91 - 25/9/91	631 777	8 2 or 3	dragline corridor Boxcut B	8
26/9/91	631 777	8 2 or 3	N-S haul road Boxcut B	9
27/9/91	777	2 or 3	N-S haul road	9
30/9/91	777	2 or 3	unknown	9
1/10/91	777	2 or 3	general	9
2/10/91 - 3/10/91	777	2 or 3	N-S haul road	9
4/10/91	777	2 or 3	office area	9
7/10/91 - 12/10/91	777	2 or 3	Boxcut B	10
14/10/91 - 16/10/91	777	2 or 3	Boxcut B	10
17/10/91 - 18/10/91	777	2 or 3	Boxcut A	10
21/10/91 - 24/10/91	777	2 or 3	Boxcut A	11
25/10/91 - 26/10/91	777	2 or 3	general	11
28/10/91 - 29/10/91	777	2 or 3	general	12

13. Typically the subsoil was placed in approximately 0.5m thick layers by motor-scrapers or up to 1m thick layers with dump trucks. There was no planned compaction. Generally each layer was subject to more than one pass due to the movement of the earth moving vehicles which included a Komatsu D155 and/or a Caterpillar D8 dozers though the wheeled vehicles would tend to follow each others tracks. The subsoil tends to retain its original density because of the manner by which it was excavated and placed.

DESIGN OF SUBSOIL MOUNDS

14. Subsoil mounds are normally built within guidelines such as those given above. This is no different from techniques used in the design of highway embankments in which side slopes are specified for typical materials. However, there are instances in which it may be necessary to undertake

further studies and in those cases there is a need to select parameters.

i. If there is a risk to adjacent property or services it is necessary at the design stage to undertake a stability analysis to ensure that in the event of a failure there will be no damage to these structures and that the side slopes give an adequate factor of safety.

ii. It is also necessary to determine the ground profile following construction of a mound since the settlement could affect adjacent services.

iii. If the mound is adjacent to a proposed excavation area it is necessary to undertake stability analyses to determine not only the safety of the mound but also the safety of the excavation face.

iv. Mounds sometimes fail because the foundation soils contain layers of laminated clay. Failures may be associated with periods of heavy rainfall.

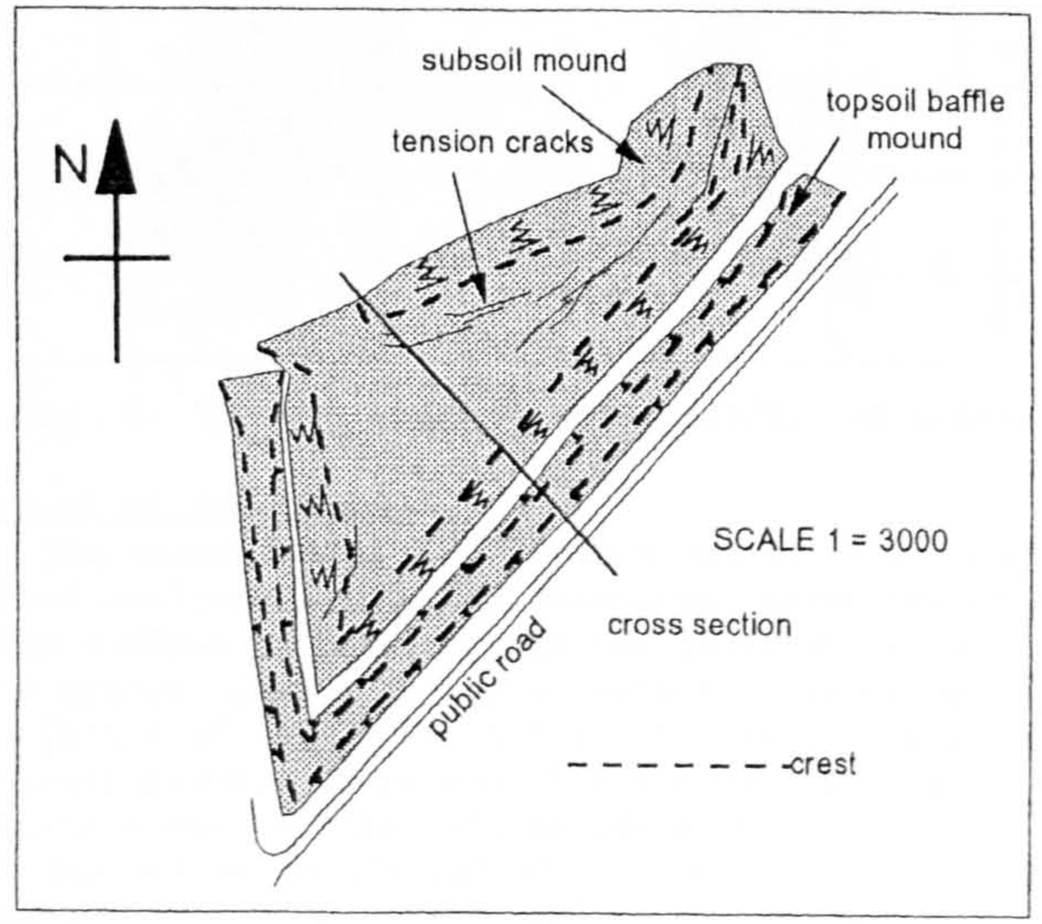

Fig. 3. Plan view of the Acklington subsoil mound showing areas of slips

15. In order to undertake these analyses it is necessary to make some assumptions about the parameters for the subsoil and the effect the mound has upon the foundation soils. There is a

difficulty as is demonstrated by the back analysis of a failed mound.

ACKLINGTON SUBSOIL MOUND

The site and soil properties

16. Fig. 3 shows a plan view of the Acklington subsoil mound, which was located in the south west corner of the site. The rectangular shape mound was bounded on two sides by a topsoil mound and on the third side by the excavation.

17. The 13m high subsoil mound was completed by July 1981. Failures occurred during construction but these were contained within the site. Damage to the edge of the road occurred about four weeks after completion. Movements between the subsoil and topsoil mounds continued but these were not catastrophic. However, there was a need to check the overall stability both in the short term and the long term to ensure that no further damage occurred to the road.

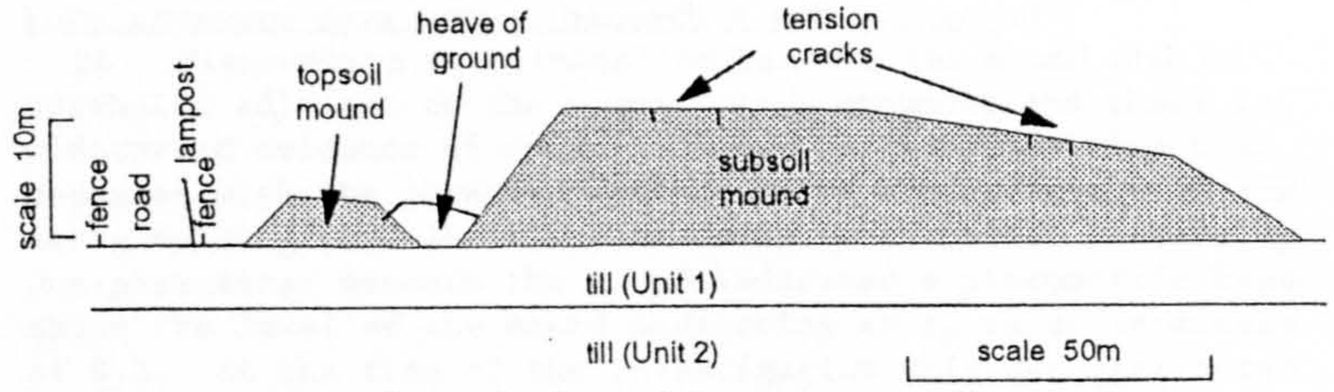

Fig. 4. Section through the Acklington subsoil mound

18. A typical section through the two mounds is shown in Fig. 4. A site investigation was carried out to determine the ground profile and parameters, the properties of the fill and to estimate the pore pressure regime.

19. Fig. 5 shows the typical soil profile and results of classification tests. Robertson et al (ref. 2) identified four Units of glacial materials in the Northumberland coalfield. All four Units, which together form a lodgement till, occur on this site. Unit 1 is a weathered ablation till, Unit 2 is a red brown ablation till, Unit 3 is a grey basal till and Unit 4 is a laminated clay. The laminated clay can occur at any depth but on this site it lies below the weathered till. Table 3 summarises the data for these soils. The figures in the brackets refer to typical values for those soils taken from an extensive database from the region.

20. The classification data for the subsoil were similar to the natural soils. The subsoil can contain all four Units of glacial soils therefore the classification data represent average values. The undrained shear strength of the subsoil was similar to the underlying soils confirming that the strength of the fill is unaffected by remoulding during placing.

Table 3 Classification and strength data for soils at Acklington mound

Soil	Water Content %	Liquid Limit %	Plastic Limit %	Undrained Shear Strength kPa	Angle of Friction
subsoil	19	40	17	90	(30)
mottled clay (Unit 1)	18(17)	39(33)	17(10)	124(150)	(30)
laminated clay (Unit 4)	25(20)	45(46)	21(26)	80(100)	(29)
red brown till (Unit 2)	16(14)	37(34)	17(14)	150(180)	(28)
grey till (Unit 3)	12(12)	30(32)	14(15)	305(200)	(31)

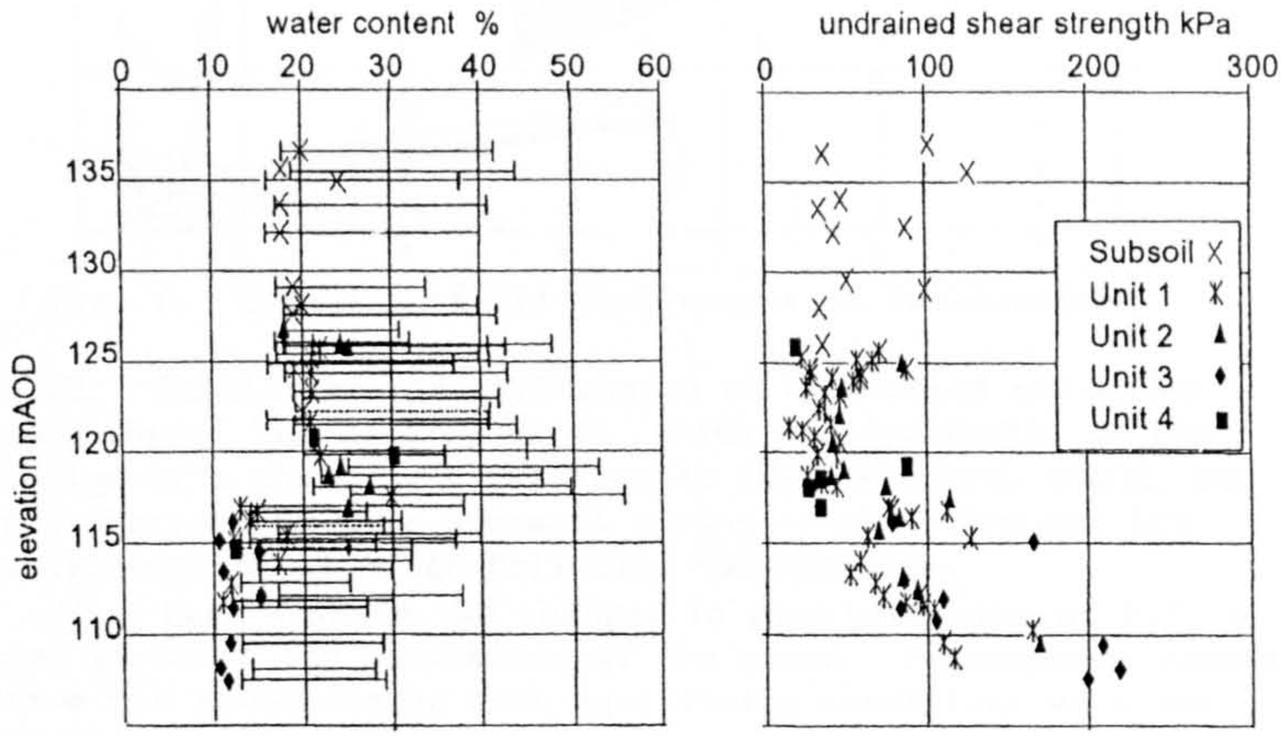

Fig. 5. Typical classification profile at Acklington

Assessment of the failure

21. The mound failed within one month of construction. An undrained analysis, using the data given above and constraining the slip surface to pass through the location of the mapped tension cracks, gave a factor of safety in excess of 2. The lowest factor of safety was for a circular slip wholly within the subsoil mound. There was little evidence to show that the slip surface was confined within the mound.

22. Inspection of the records of construction suggests that toe heave between the topsoil and subsoil mounds occurred during construction, perhaps due to the foundation soils being squeezed out. Further heave between the mounds occurred once construction was complete. These were accompanied by tension cracks forming in the surface of the subsoil mound. Failure did not occur as a result of increased precipitation since, at the time, rainfall levels were relatively low.

23. About one month after construction heave was noticed outside the site boundary adjacent to the topsoil mound. There was no evidence of failure of the topsoil mound which suggests that the failure of the subsoil mound and the associated heave between the mounds caused the topsoil mound to be moved bodily. This mechanism would have resulted in heave beyond the topsoil mound.

24. Further movements of the subsoil mound on the other two sides occurred some months later. While these did not give cause for immediate concern there was the fear that they could affect the proposed excavation face.

25. A fully drained analysis would show that the subsoil mound was unsafe since the side slopes were greater than the angle of friction. Any failure surface passing through the foundation soils would give a factor of safety in excess of 1 in the longer term.

Pore pressure development beneath a subsoil mound

26. Piezometers were installed beneath the mound and in boreholes adjacent to the mound. In Northumberland there is widespread evidence of under drainage to the underlying Coal Measures with the phreatic surface being approximately at one metre below ground level but it tends to fluctuate seasonally. One piezometer beneath the mound indicated a piezometric head above the level of the mound suggesting an r_u value in excess of 0.5. At the time of the investigation this was attributed to instrument error because the values were much in excess of those anticipated based on published data.

27. It is generally accepted that the maximum pore pressure generated is about 40% of the increase in total vertical stress (ref. 3). However, if this is applied to the foundation soils and the subsoil mound, it produces factors of safety significantly less than 1 in the short term.

COLLIERSDEAN SUBSOIL MOUND

28. The Colliersdean subsoil mound was constructed as described above. It is about two miles from Acklington and founded on similar glacial soils. Given the speed with which these mounds are built and the size of equipment used, it was considered unwise to attempt to instrument the mound itself.

29. The depth to the Coal Measures is 14m, and all four Units of glacial soils are present. Piezometers, settlement cells and load cells were placed within the foundation soils at locations shown on Fig. 6. Additional piezometers and settlement cells were placed some distance from the mound to establish equilibrium conditions. This was necessary because of the limited time between installation and construction of the mound.

30. The instruments were recorded automatically, the interval between readings ranging from two hours to twelve

hours. During construction surface profiles were taken daily using levels.

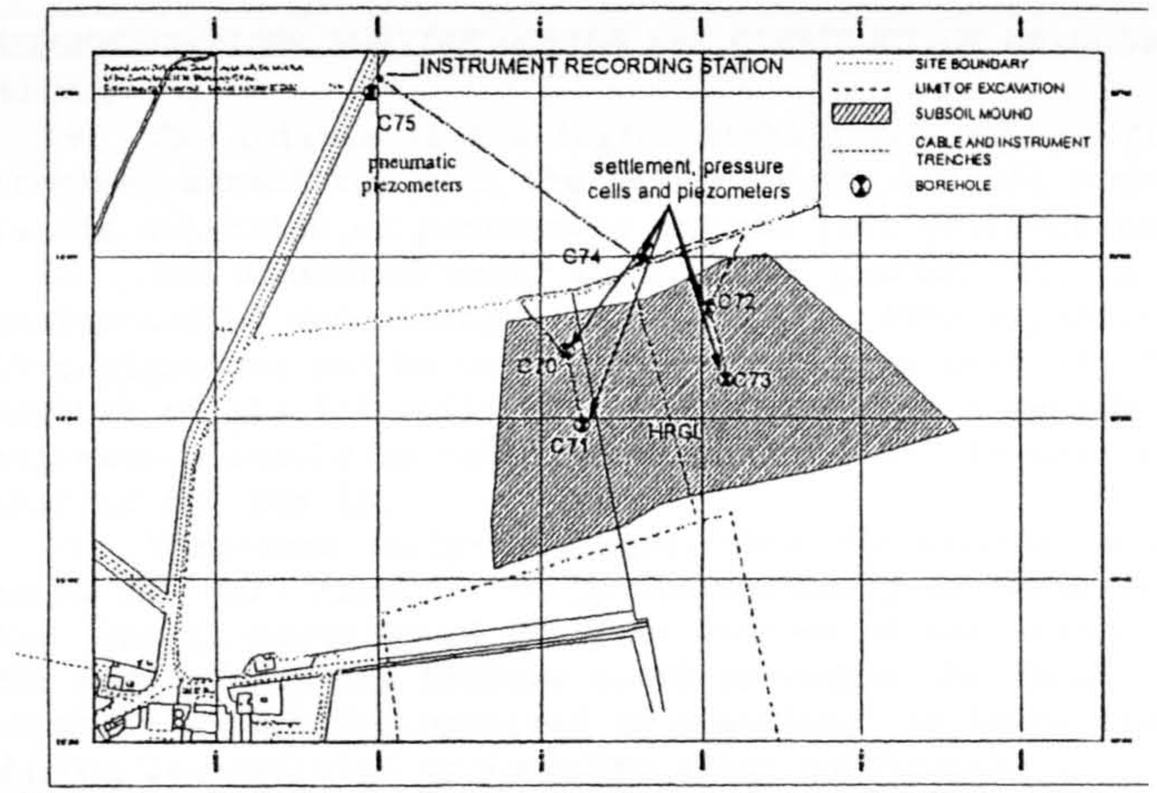

Fig. 6. Location of the instruments at Colliersdean

31. Details of the performance of this mound are to be published. The data obtained, which are applicable to the design of a mound, and relevant to the Acklington mound, are the increase in pore pressure during construction and its subsequent dissipation following construction.

32. Fig. 7 shows the changes in pore pressures at 1.7, 4 and 6m below the centreline of the mound. Piezometers remote from the mound showed that equilibrium conditions were not reached by the end of construction thus the changes shown in Fig. 7 are likely to include some increase in pore pressure due to the ambient conditions. The maximum increase in pore pressure above the ambient conditions after correcting for the time to reach equilibrium was 174, 97 and 32 kN/m^2 for depths 6, 4 and 1.7 m respectively. This is equivalent to r_u factors of 0.60, 0.40 and 0.16.

33. It shows that the greatest increase in pore pressure occurs at depth. There may be two reasons for this. The shallow piezometer was placed in a trench which had to be 1.5m deep because of possible damage to the cables during the mound construction. The backfill material is likely to be more permeable than the surrounding soil therefore the trench could act as a drain. The upper till (Unit 1) is weathered and it is likely that the permeability is greater than the unweathered till below.

34. After 1.3 years the excess pore pressures were 115, 68 and 27 kN/m^2 at the same locations (that is r_u factors of 0.46, 0.30 and 0.14).

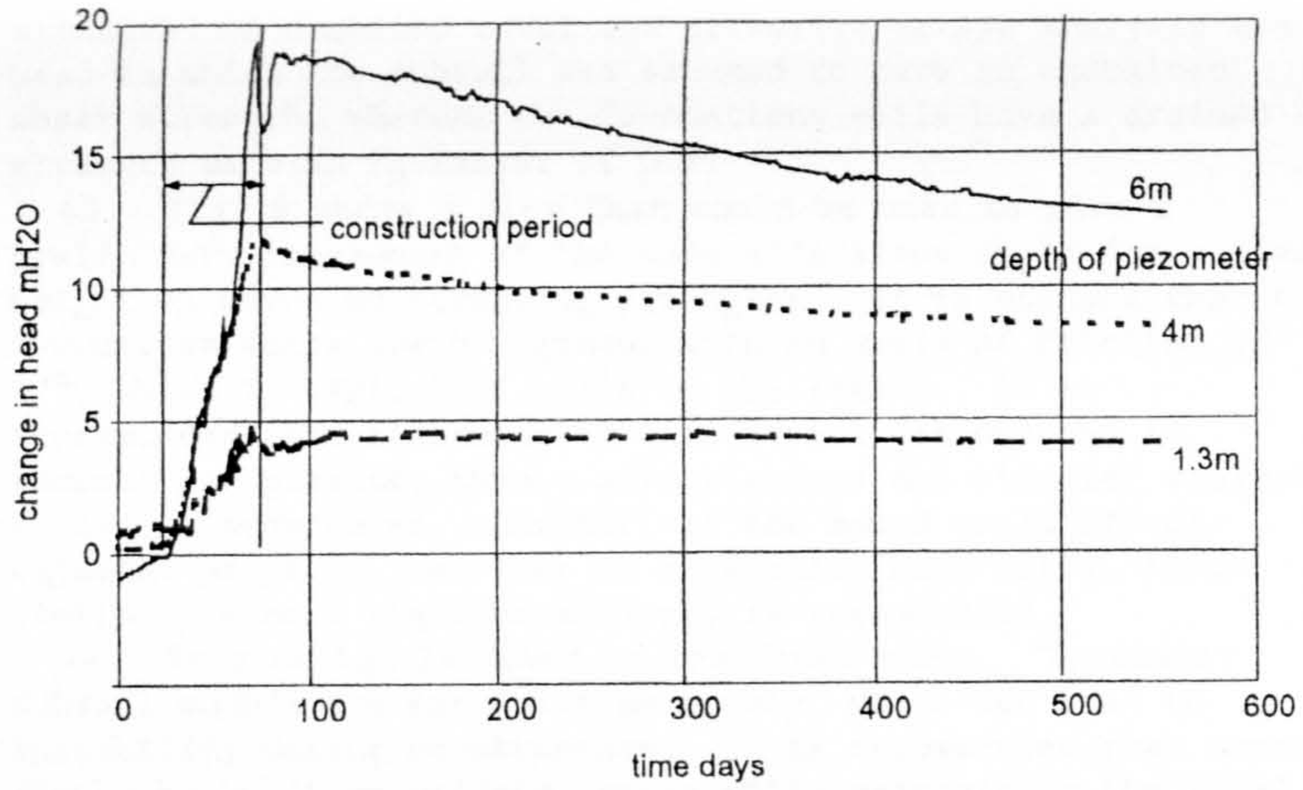

Fig. 7. The change in head of water during and after construction beneath the centre of the mound

THE INSTABILITY OF THE ACKLINGTON MOUND

35. The pore pressure data from Colliersdean suggests that it is possible to develop large pore pressures within the foundation soils. It was not possible to determine the pore pressure changes within the subsoil because of the difficulties of ensuring that instruments would be unaffected by construction.

36. Further analyses of the Acklington mound were undertaken assuming undrained conditions in the subsoil using the measured shear strengths and partially drained conditions in the foundations soils with r_u factors of up to 0.7. This combination of total and effective stress analysis may not be totally satisfactory but the alternative is to develop a peizometric distribution based upon an assumed model for the development of excess pore pressure and its dissipation. It is considered that the simple assumptions given above are adequate.

37. A circular stability analysis showed that the critical circle passed through the position of the mapped tension cracks and the toe of the topsoil mound which conforms with the observations. An r_u of 0.7 gives a factor of safety of 1.13. The formation of a tension crack and infilling with water both act to reduce this factor. The critical circle passes through the weathered till (Unit 1).

38. It is proposed that the failure at Acklington occurred in two stages. The first stage was the failure of the subsoil mound which produced heave between the mounds. The second resulted from the horizontal force of the failed subsoil mound

acting upon and causing movement of an otherwise stable topsoil mound. This, in turn, resulted in damage to the public road.

RECOMMENDATIONS FOR THE DESIGN AND CONSTRUCTION OF SUBSOIL MOUNDS

39. The failure at Acklington highlighted some of the problems associated with the design of the subsoil mounds, that is the selection of parameters and the pore pressure profile.

40. The undrained shear strength of glacial till is usually unaffected by remoulding. Therefore data from exploratory site investigations can be used. The subsoil can consist of a mixture of glacial soils but, for design, the strength parameters should be taken as those of Unit 1 in most cases, that is the top 2m.

41. Undrained analyses suggest that the Acklington subsoil mound was very stable. Fully drained analyses indicate that the subsoil mound would fail but because of the steepness of the side slopes the failure would be within the mound. Neither of these situations occurred in practice. Failures can occur during construction or sometime after construction.

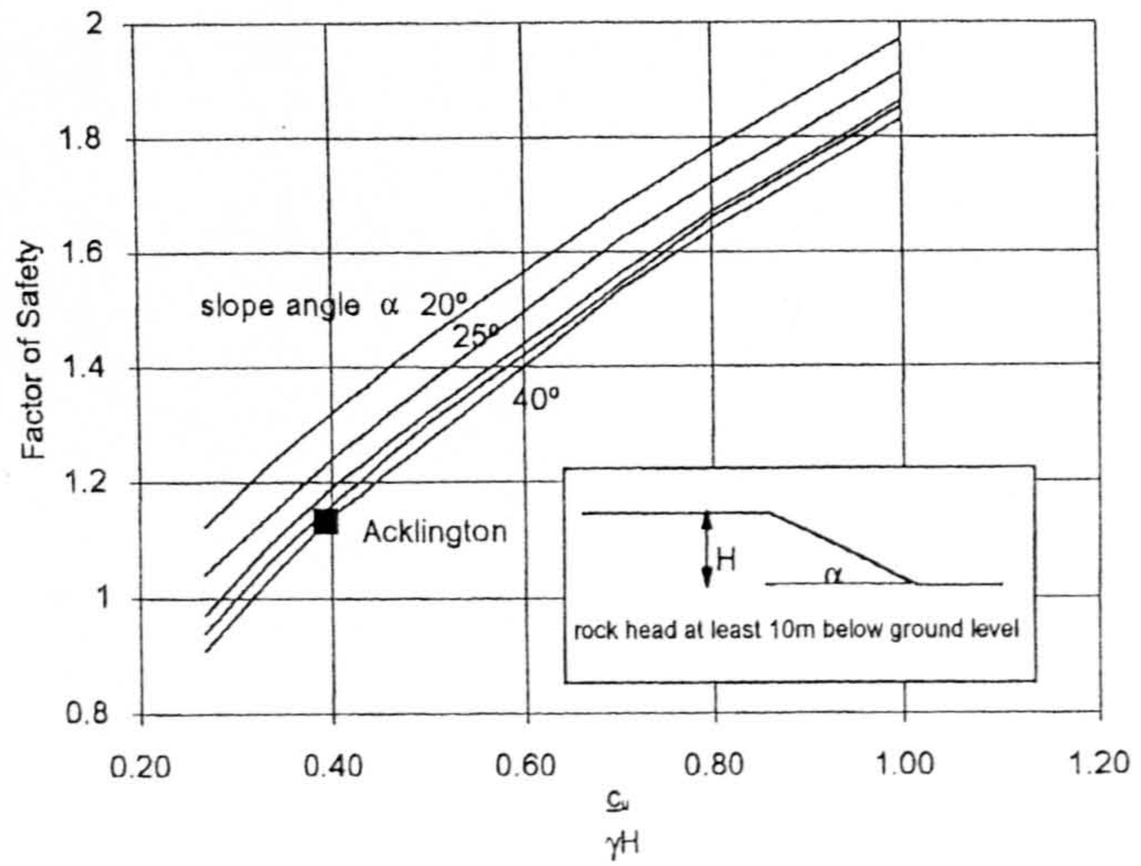

Fig. 8. A chart to assess the side slopes of a subsoil mound give the height of mound and the undrained shear strength of the subsoil

42. Evidence from an instrumented mound at Colliersdean shows that substantial pore pressures develop within the foundation soils, perhaps up to 0.7 of the overburden pressure. A simple design approach is suggested with the assumption that the strength of the subsoil is the same as the in situ

strength. A combined total and effective stress analysis was used in which the subsoil was assumed to have an undrained shear strength, whereas the foundations soils have a drained strength with an r_u factor of 0.7.

43. Fig. 8 shows a plot that could be used to give a preliminary assessment of the safe side slope angle for a given height of mound of differing strengths. It is assumed that the foundation soils are homogenous with an angle of friction of 30°, which is typical of tills in the region. It is recommended that if laminated clay layers (as opposed to lenses) are present, then a more rigorous non circular analysis should be undertaken. Further, if the mound could affect adjacent property, services or excavation high walls, then similarly a more rigorous analysis is recommended.

44. This design is based on the final shape. Typically subsoil mounds are not built uniformly, which can lead to instability during construction. It is recommended that mounds should be built in uniform layers while maintaining the final side slopes. This will reduce the possibility of instability.

CONCLUSIONS

45. It has been established that excess pore pressures developed in glacial tills following the rapid construction of a subsoil mound, are in excess of those expected from conventional construction of engineered fills.

46. It has been confirmed that the undrained shear strength of a subsoil mound is similar to the in situ strength of the source glacial materials.

47. A simple design procedure has been suggested that will allow preliminary estimates to be made of suitable side slopes for a given height of mound.

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PAPER C15 HORSLEY 2

Charles, J.A., Burford, D. and Hughes, D.B. 1993.

Settlement of opencast coal mining backfill at Horsley, 1973-1992.

Proceedings of 14th International Conference on Engineered Fills, Newcastle upon Tyne (eds. C.J.F.P. Jones, B.G. Clarke, A.I.B. Moffat), Thomas Telford Ltd., 429-440.

34. Settlement of opencast coal mining backfill at Horsley 1973–1992

J. A. CHARLES and D. BURFORD, Building Research Establishment, UK, and D. B. HUGHES, British Coal Opencast, UK

SYNOPSIS. A 70 m deep mudstone and sandstone opencast coal mining backfill at Horsley near Newcastle has been monitored throughout a 20 year period from 1973 to 1992 during which the ground surface has settled by as much as 0.8 m. A 34 m rise in ground water level between 1974 and 1977 has been a principal cause of the settlement. Settlement at depth within the fill has been measured using magnet extensometers and surface movements have been measured by precise levelling. The implications of the ground movements for building developments on opencast backfills are briefly discussed.

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INTRODUCTION

1. The effect of a rising ground water level on the settlement of an uncompacted opencast coal mining backfill has been monitored at Horsley in the north east of England. The investigation has been carried out jointly by the Building Research Establishment and British Coal Opencast. Interim reports were presented in 1977 and 1984 (Refs 1 and 2). Monitoring at the site was discontinued at the end of 1992 and this paper presents a final report on the investigation.

2. Backfilling took place between 1961 and 1970 with restoration completed in 1973. It was necessary to de-water the site during opencast mining and pumping continued for some time after the completion of backfilling. Instrumentation was installed in 1973 to monitor settlement at different depths within the backfill. When pumping stopped in April 1974, the ground water level rose 34 m reaching a new equilibrium level in 1977. The site was returned to agriculture after the completion of backfilling and, so far as is known, no building development is planned.

3. The opencast workings covered an area about 1500 m long by 600 m wide and the backfill has a maximum depth of almost 70 m. The excavated strata belong to the Middle and Lower Coal Measures of the Carboniferous system. In the upper part of the workings excavation of the overburden was carried out by face shovels and backfilling was by end tipping from dump trucks. In the lower part of the workings excavation was by dragline.

4. Information about the backfill was obtained in 1973 when boreholes for the installation of instrumentation were drilled using a rotary air flush rig. The heterogeneous fill was composed principally of mudstone and sandstone fragments, with mudstone predominating. Less than 10% of the backfill was boulders. Cavities up to 0.5 m deep were found. Open drive 100 mm diameter samples indicated that about 10% of particles were finer than 0.075 mm and about 50% were coarser than 2.36 mm. The average dry density was 1.70 Mg/m^3 and the average moisture content was 7%, but these properties showed great variations between samples. The degree of saturation of the fill as placed varied between 10% and 100%. The relative density (I_D) was estimated to be about 60% and the mean SPT N value of 29 confirmed that the fill was of medium relative density. A field test in a borehole at the site of the lagoon indicated a permeability greater that 10^{-4} m/s.

INSTRUMENTATION

5. The following types of field measurement have been made:

(a) settlement at different depths within the fill has been monitored using 5 magnet extensometers installed in boreholes drilled through the full depth of fill,
(b) surface settlement of the fill has been monitored by precise levelling of traverses of surface settlement stations,
(c) ground water level has been monitored in standpipe piezometers.

6. Late in 1973 five magnet extensometers were installed in 0.15 m diameter boreholes drilled through the backfill. These are referred to as borehole settlement gauges and details of the installation have been given in ref 1. The bottom magnet marker of each gauge was installed in bedrock and formed a stable reference point. From measurements taken on successive occasions, settlement of the magnets relative to the reference magnet could be computed. In this way settlement has been monitored at different depths within the backfill.

7. The locations of the five borehole gauges, which were selected to provide as much information as possible about the behaviour of the backfill, are shown in Figure 1. Information about the gauges is summarised in Table 1. Gauge D1 was installed in fill which had been preloaded by an overburden heap with a maximum height above restored ground level of 30 m and gauge C11 was installed in an old lagoon area. Gauge A9 was installed in the oldest fill and gauge D15 in the most recently placed fill.

8. A traverse of surface settlement stations was established adjacent to each borehole gauge to supplement the information on surface settlement. Precise levelling has been carried out from bench marks established on undisturbed ground outside the limits of the opencast workings. Traverse E, in an area of intermediate age, had no borehole settlement gauge.

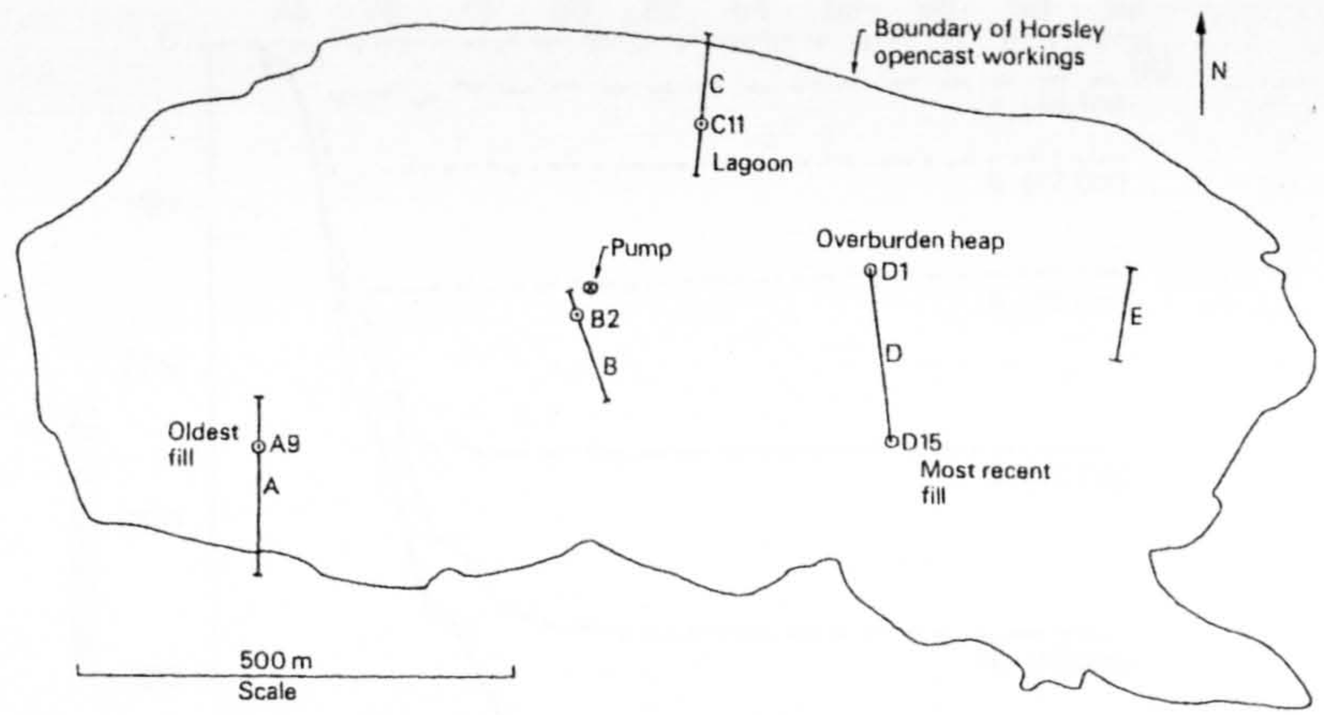

Figure 1. Plan of Horsley site showing location of magnet extensometers and traverses of surface settlement stations

Table 1. Ground conditions at magnet extensometers

Gauge no	Ground level	Rock-head	Fill depth	Fill date	Inundated depth	Fill condition
	mAOD	mAOD	m		m	
A9	98.6	38.0	60.6	1961	46	oldest
B2	101.8	38.7	63.1	1964	45	deepest
C11	94.9	49.2	45.7	1965	35	lagoon
D1	108.1	52.6	55.5	1966	31	preloaded
D15	119.2	72.7	46.5	1970	11	recent
E12[1]	115.8	68	48	1966	17	intmdt age

[1] There was no magnet extensometer at E12, but it is listed because the maximum settlement was recorded at this location.

MONITORED BEHAVIOUR OF FILL

9. The surface settlements monitored by precise levelling at the five borehole gauges are plotted in Figure 2. The maximum movement measured anywhere within the site is at surface settlement station E12 where the fill has settled 0.8m and this is also plotted in Figure 2. The smallest movement measured at a borehole settlement gauge has occurred at gauge D1 and this can be attributed to the effect of preloading by a 30 m high overburden heap. The settlement at gauge C11 has also been small but it should be noted that, prior to the rise in water level, the settlement rate at this gauge was greater than in the other parts of the site. It may be that in the years before monitoring commenced the settlement of the lagoon area, which is composed of a wet and more cohesive fill, was large.

10. Figure 3(a) shows the settlement measured at different depths in the fill at gauge B2 where the fill is deepest and

Figure 3(b) shows the rise in ground water level plotted to the same time scale. A surface settlement of 0.5 m has been measured at this gauge and vertical compression has occurred throughout almost the full depth of the gauge. Figure 4 shows the total settlement measured during the 20 year period plotted against depth for gauges B2, C11, D1 and D15.

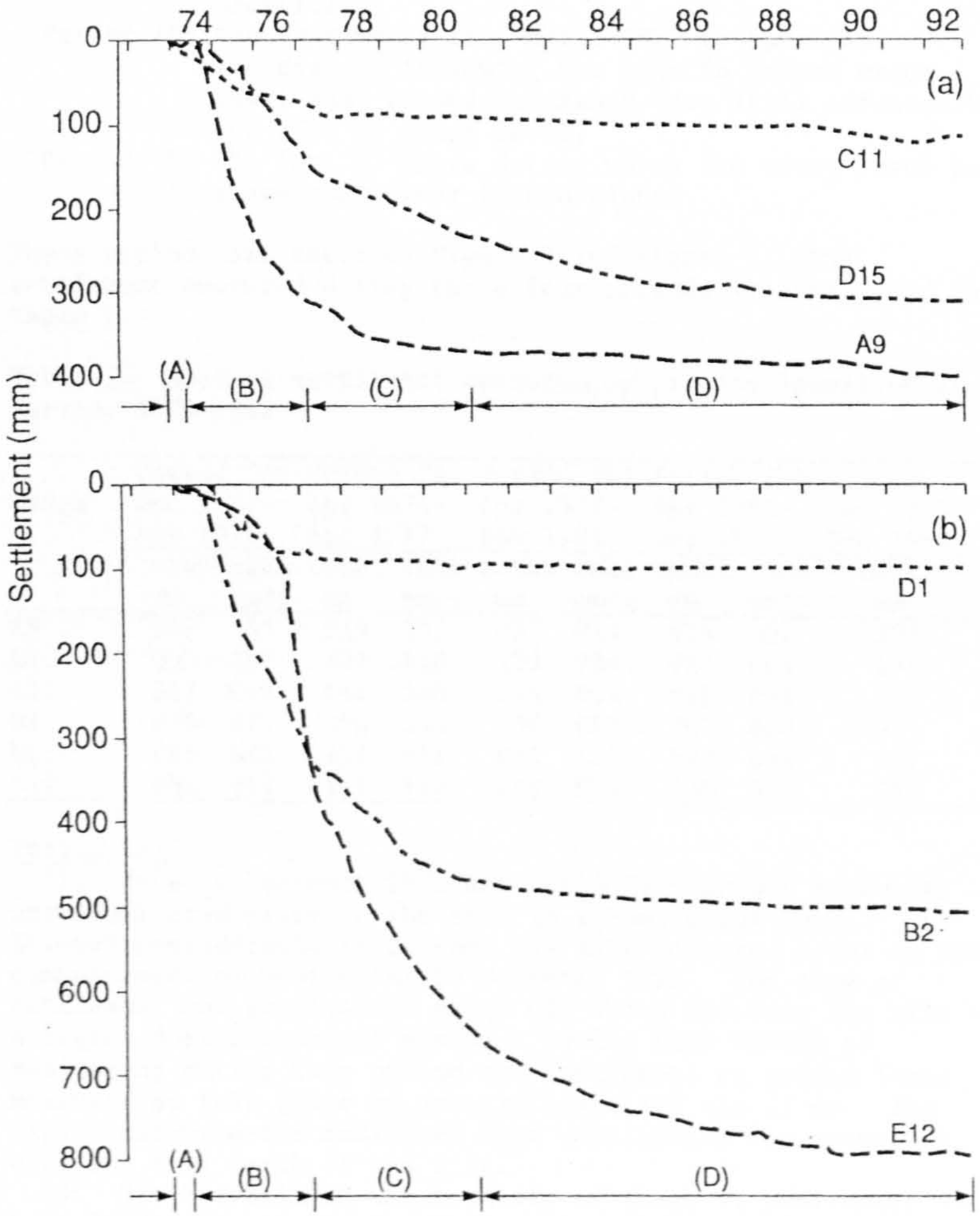

Figure 2. Development of surface settlement during successive periods (A), (B), (C) and (D):
(a) settlement at locations A9, C11, D15,
(b) settlement at locations B2, D1, E12.

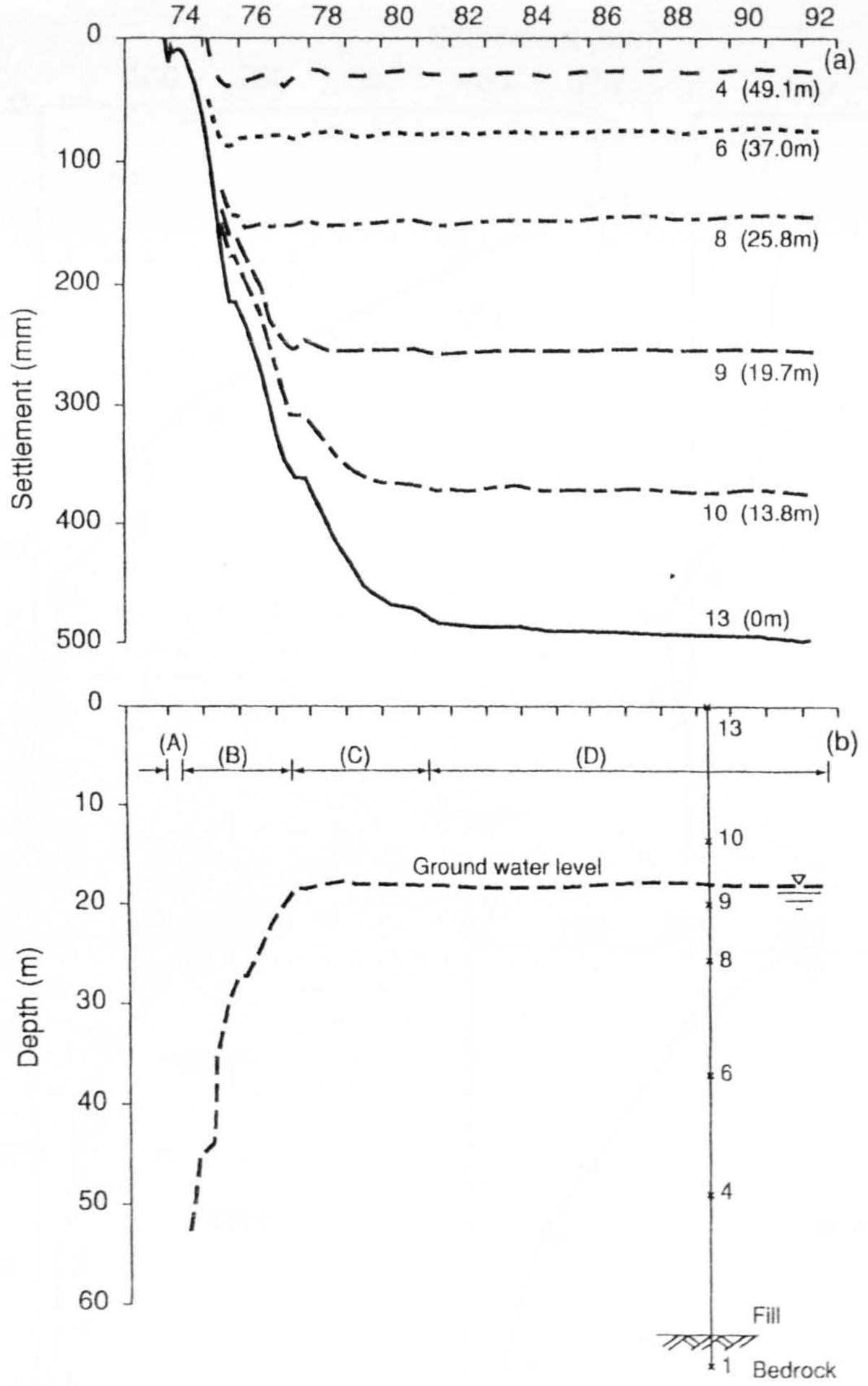

Figure 3. Relationship between settlement at different depths within the deepest fill (gauge B2) and the rise in ground water level:
(a) settlement plotted against time for selected magnet markers, depth of magnet marker is shown in () after reference number of magnet eg 8 (25.8 m),
(b) ground water level versus time with depths of magnet markers shown in relation to ground water level.

11. It is helpful to examine the settlement of the backfill during four periods:

Period (A) the few months prior to April 1974 when settlement was monitored while pumping kept the water level down in the bedrock below the backfill,
Period (B) the three years from April 1974 to April 1977 when the ground water level rose some 34 m through the backfill,
Period (C) the four years from April 1977 to April 1981 immediately following the rise in ground water level when ground movements were still affected by the rise in water level,
Period (D) the last 12 years during which the water level has shown only minor fluctuations.

These periods are shown on Figure 2 and Figure 3. The settlement measured during these four periods is summarised in Table 2.

Table 2. Surface settlement measured by precise levelling at Horsley 1973-1992

Gauge	Period (A) Dec 1973- Apr 1974		Period (B) Apr 1974- Apr 1977		Period (C) Apr 1977- Apr 1981		Period (D) Apr 1981- Dec 1992		Dec 1973- Dec 1992
	total mm	rate mm/y	total mm	rate mm/y	total mm	rate mm/y	total mm	rate mm/y	total mm
A9	002	005	310	103	057	014	028	002	397
B2	002	006	331	110	133	034	033	003	499
C11	017	043	061	020	015	004	016	001	109
D1	-010	-021	098	033	006	002	001	000	095
D15	001	002	152	051	081	021	073	006	307
E12	008	017	354	118	288	074	138	012	788

Period (A)

12. Between December 1973 and April 1974 ground movements were monitored prior to the rise in ground water level. However considerable settlement may have occurred prior to the commencement of monitoring in December 1973. The rate of settlement was greatest at gauge C11 which had been the site of a lagoon during opencast mining. In the four months of monitoring during this period the settlement at ground level measured at this gauge by precise levelling was 17 mm. The magnet extensometer indicated that compression was occurring over the full depth of the fill.

13. The backfill in the locality of gauge D1 previously had been loaded by a large overburden heap with a maximum height above restored ground level of 30 m. This had been removed two years before the measurements began. Four months of precise levelling prior to April 1974 showed a heave of 10 mm at ground level. The magnet extensometer indicated that this movement was caused by expansion at the base of the fill.

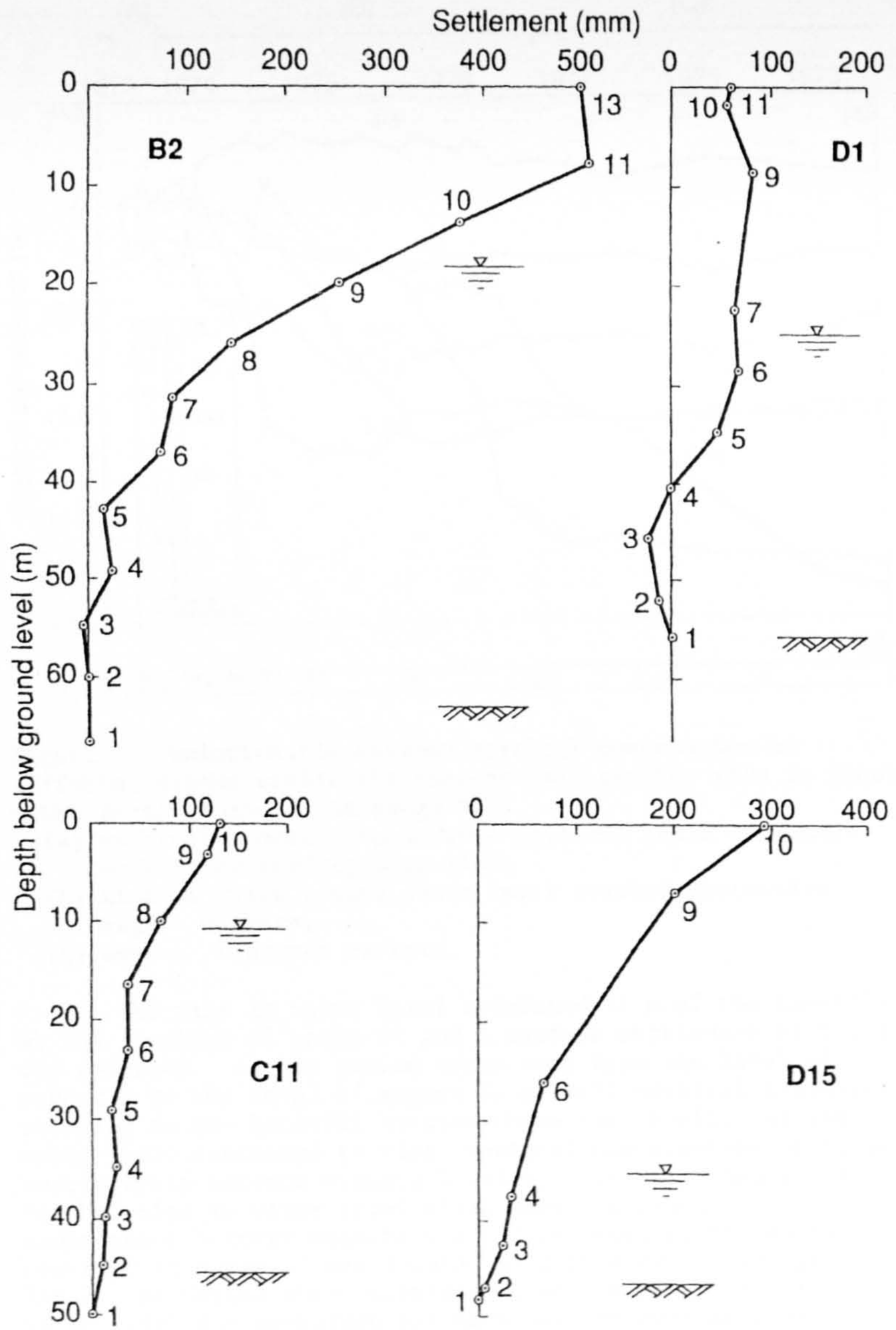

Figure 4. Variation of settlement with depth within the fill measured during period 1973-1992 at four locations B2, C11, D1, D15; reference numbers of magnet markers are given for each settlement measurement.

14. At gauge B2 close to the pump the water table varied between 5 m and 10 m above rockhead during this initial period in which pumping continued. The settlement measured by precise levelling was 2 mm during the four month period.

Period (B)

15. Ground movements were monitored from April 1974 to April 1977 as the ground water level rose. The submergence of a fill by a rising ground water level increases pore pressures thus reducing effective stresses and consequently heave of the ground might be expected. In practice most partially saturated fills undergo a reduction in volume, termed collapse compression, when their moisture content is increased. Partially saturated fills are susceptible to collapse compression under a wide range of applied stress when first inundated if they have been placed in a sufficiently loose and/or dry condition. Thus saturation often causes settlement due to collapse compression.

16. The ground water level at gauge B2 rose by 20 m between April 1974 and April 1975, 9 m in the following 12 months and 5 m in the 12 months after that. From June 1975 onwards the water level measured in the five borehole gauges has been virtually the same height above OD. Having reached a new equilibrium water level in April 1977 at about 83 mAOD subsequent fluctuations in water level have been small. A maximum water level of 84 mAOD was recorded in August 1978. The final equilibrium level of the ground water level in the opencast backfill appears to have been controlled largely by the topography of the site.

17. At gauge B2, where the fill was deepest, 0.33 m settlement occurred at the ground surface during this 3 year period as the water level rose 34 m. Figure 5 shows the relationship between the rise in ground water level and the vertical compression of the backfill. In Figure 5(a) the vertical compression between adjacent magnet markers in the borehole settlement gauge is plotted against time. Figure 5(b) indicates with arrows the dates at which the ground water reached the level of successive magnet markers. Figure 5(c) shows the positions of the magnet markers in the borehole settlement gauge. As the ground water rose from the level of magnet 5 to the level of magnet 6 a vertical compression of just over 1% occurred over the depth of fill between these two magnets. Then as the ground water continued to rise large compressions occurred successively between magnets 7 and 8, 8 and 9, and, 9 and 10. The effect of saturation in producing collapse compression within the backfill was clearly demonstrated. Locally compressions as large as 2% were recorded but the average compression over the 34 m depth of backfill saturated by the rising water table was smaller than 1%. It should be noted that at an early stage a small expansion occurred between magnet markers 4 and 5 and that little compression occurred between magnets 6 and 7.

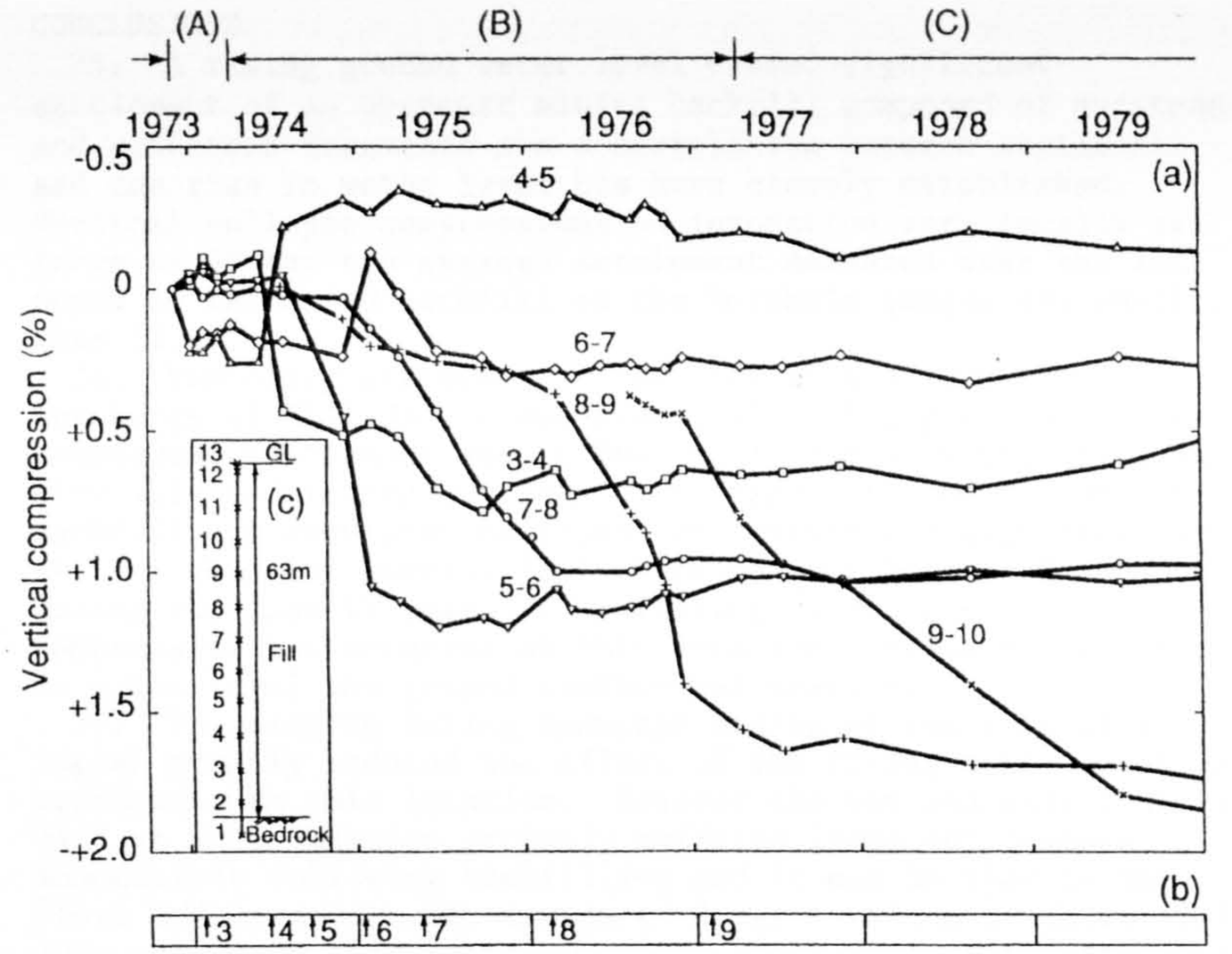

Figure 5. Relationship between vertical compression at different depths within the deepest fill and the rise in ground water level (measured at gauge B2):
(a) vertical compression between adjacent pairs of magnet markers plotted against time,
(b) time at which ground water level reached successive magnet markers,
(c) depths of magnet markers.

18. The rise in water level saturated 31 m of the backfill at the location of gauge D1 and a surface settlement of 0.1 m was observed. As the ground water rose from the level of magnet 2 to the level of magnet 3, a small vertical expansion occurred in the backfill between these two levels. As the water level continued to rise, vertical compressions occurred successively between magnets 3 and 4, 4 and 5, 5 and 6. A further rise in water level above magnet 6 caused no compression between magnets 6 and 7 and most of the settlement observed at gauge D1 was located more than 28 m below ground level. At depths where compression was caused by the rising water level the magnitude was much smaller than at gauge B2 and this can be attributed to the effect of the preloading produced by the overburden heap.

19. The settlement at C11 gave little indication of being affected by the rising water level. As this area was the site of a lagoon during opencast working, it is probable that the backfill was sufficiently wetted at that stage to prevent

further settlement occurring due to the rising ground water level. Only when the water level rose from the level of magnet 5 to the level of magnet 6 did compression occur that was clearly associated with the rise in water level. Precise levelling recorded 61 mm settlement during this period. At this gauge settlement occurred fairly uniformly through the full depth of the backfill.

20. The rising ground water level had little effect on settlement at gauge D15 because the gauge is situated on high ground and only the bottom 11 m of the backfill have been inundated (Table 1). Between April 1975 and April 1977 as the ground water inundated the bottom 11 m of the backfill, there was some increase in the rate of settlement.

Period (C)

21. Ground movements were monitored in the period immediately following the rise in ground water level between April 1977 and April 1981. At A9, B2, D15 and E12 the rate of settlement was between three times and ten times as large as the rate prior to the rise in ground water level. Large movements continued at B2, but Figure 2 shows that the continuing settlement was largely caused by compression of the fill above the ground water level. Precise levelling has shown that the greatest movement during this period occurred at E12 where 0.29 m settlement was observed. The reason for such large movements at this location is not known.

Period (D)

22. Ground movement monitoring continued in the period from April 1981 to November 1992 which commenced several years after the rise in ground water level. At most locations the rate of settlement during this period has been smaller than the rate prior to the rise in ground water level. The only magnet extensometer that provides an exception to this is gauge D15. This is in the most recent fill and currently the settlement rate is still greater than at the other gauges. At the other borehole gauges the rate of movement is 3 mm per year or smaller. The rate of movement at E12 is greater than at any of the five borehole gauges.

DIFFERENTIAL SETTLEMENT

23. It is differential settlement rather than total settlement which causes damage to buildings. The traverses of settlement stations provide information on differential settlement. In assessing this data it should be remembered that if a structure had been built at the location where the differential settlement has been measured, the differential movement would have been modified and probably greatly reduced by the stiffening effect of the structure.

24. The maximum differential settlement has been recorded on traverse E. At a location where the total settlement is 737 mm surface stations 5 m apart have shown a differential settlement of 146 mm corresponding to a deflection ratio of 2.1×10^{-3}.

CONCLUSIONS

25. A rising ground water level caused significant settlement of an opencast mining backfill composed of mudstone and sandstone fragments and a correlation between settlement and the rise in water level has been clearly established. Vertical collapse compressions on inundation were locally as large as 2% but the average settlement measured over the full depth of inundated backfill at the borehole gauges was smaller than 1%.

26. Temporary preloading of an area with a 30 m high surcharge of fill during opencast coal mining greatly reduced subsequent settlement due to the rising ground water level and virtually eliminated any long term creep settlement. When the backfill was saturated collapse compression was significantly smaller than for similar fill which had not been preloaded and, during the last 12 years of monitoring, virtually no compression has occurred at this location. Prior to the rise in water level the ground surface was heaving.

27. Pre-wetting during opencast mining at the site of a lagoon greatly reduced the effect of the rising water level on settlement at this location. However the wet and more cohesive fill at this location probably suffered large settlements immediately following backfilling and it may be that in the years before monitoring commenced large settlements occurred in this area. As this settlement took place at an early stage, it might be of less importance for building development.

28. Although the investigation has demonstrated the effect of a rising ground water level in producing collapse compression and has shown the relationship between some of the features of the opencast mining and subsequent settlement behaviour, the pattern of settlement of a restored opencast mining site has been found to be complex. A number of observations are difficult to explain. For example, a significant proportion of the settlement monitored at the borehole gauges was due to compression in the upper 10 m of backfill which was not saturated by the rising ground water level. It must be concluded that even with a good knowledge of the opencast operations that had occurred on the site, it would have been difficult to predict the magnitude and rate of the settlement of the backfill.

29. The investigation has considerable significance for building developments on restored opencast mining sites. The complexity of the settlement pattern measured at Horsley points to the need for careful investigation. As much information as possible should be obtained about opencast operations, and settlement and water levels should be monitored over a realistic period. In addition to settlement due to self-weight and applied loads, the possibility of collapse settlement on inundation should be addressed. The results from the Horsley investigation have shown the effects that could be caused by inundation of a partially saturated opencast mining backfill subsequent to building on the site.

ACKNOWLEDGEMENT

30. The investigations described in this paper form part of the research programme of the Building Research Establishment. The authors wish to thank British Coal Opencast for their permission to publish this paper. The views expressed herein are entirely those of the authors.

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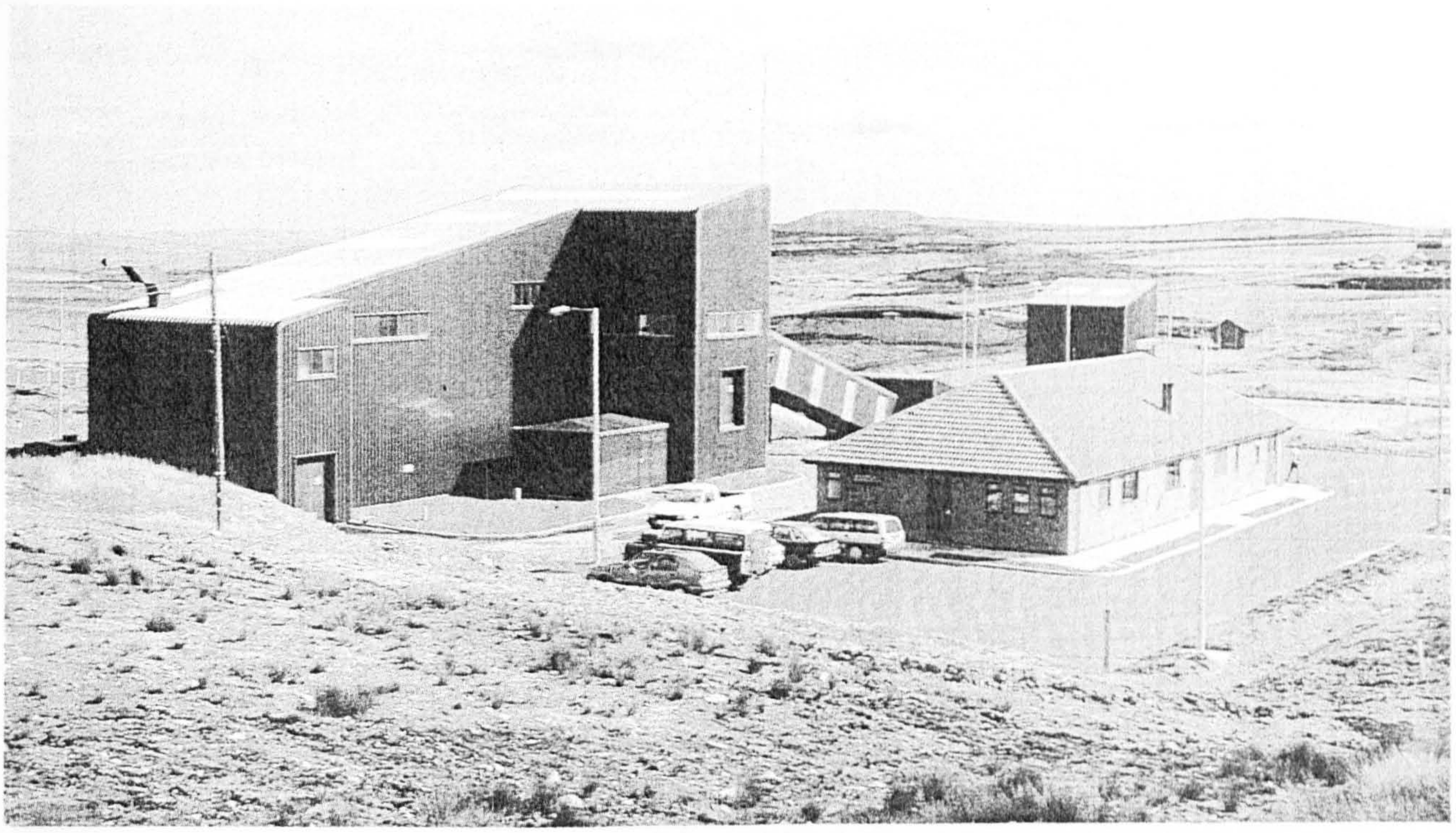


The crushing and screening plant at Plenmeller OCCS

PLENMELLER OCCS — A GROUND ENGINEERING CASE STUDY

by David Hughes and David Norbury

Plenmeller Opencast Coal Site is located in south-west Northumberland in the South Tyne Valley. It is close to the town of Haltwhistle which lies on the Carlisle to Newcastle A69 trunk road and the main-line railway, and is some 6km south of Hadrian's Wall. The southern part of the site lies within the designated North Pennines Area of Outstanding Natural Beauty.

The site occupies a depression which is situated on the north-facing slopes on the south side of the valley, where the average elevation is around 300m AOD. This east–west aligned depression lies within the catchments of the eastward-flowing Kingswood Burn and westward-flowing Level Sike. The Blackcleugh Burn and the Willimontswyke Burn both rise on the northern rim of the depression and flow north-eastwards into the river South Tyne. On the southern side of the site, the higher ground at Fellhouse Fell prevents any drainage to the south. The average annual rainfall is around 800mm.

The area is best described as upland moorland which includes areas of peat bog, mat grass, rough pasture, enclosed improved pasture, and forms part of the Plenmeller and Kingswood Commons. Much of the site has been affected by previous mining activity as evidenced by pitfallen land, many abandoned shafts and drifts, and by several small spoil heaps. Until site work began, the land was mainly used for sheep grazing and grouse shooting.

An application for authorization for the Plenmeller site was made to the Secretary of State for Energy in December 1982 under the Opencast Coal Act 1958, but was withdrawn in February 1984 as a consequence of new legislation. A new planning application was made in March 1985 under the Town and Country Planning Act 1971, which resulted in a public inquiry at Haltwhistle held during June and July 1986. In January 1987 the Inquiry Inspector reported in favour of the project, and authorization was notified in June 1987, with many detailed planning conditions attached; however, the Mineral Planning Authority (Northumberland County Council) appealed against the decision to the High Court. This hearing took place in February 1989, and confirmed the authorization and conditions as per the June 1987 notification.

The working of this opencast coal site involves many operations including the construction of lagoons, construction of a permanent highway diversion, excavation and storage of peat in bunded areas, excavation and storage of topsoils in mounds, excavation and backfilling of cuts to expose and recover coal. The coal is crushed and screened on site and then transported by overland conveyor to a railhead loading facility located on the Newcastle to Carlisle main line.

Preliminary construction works for coal processing and transportation facilities commenced in August 1989, and the main site contract followed in March 1991. The first coal was produced in August 1991. Some site statistics are as follows:

Coal tonnage in contract	2,034,514 tonnes
Overburden-to-coal ratio	19.7 to 1
Site area	458 hectares
Excavation area	190 hectares
Maximum volume of spoil stored above ground	8,000,000m³
Expected coaling life of site	8 years

The coal itself is generally of good quality, with a high calorific value, and is primarily consumed by the power generators,

Mr Hughes was formerly with BCO, Mr Norbury is with Soil Mechanics Ltd and acts for Mowlem Mining, the main contractors.

A section of the working area showing evidence of groundwater seepage

although other industrial and export markets are also served.

GROUND INVESTIGATIONS

Geotechnical Codes of Practice for spoil mound stability[1] and excavated slope stability[2] were introduced into the UK opencast coal mining industry in 1982 and 1989 respectively. The Geotechnical Stability Report, with all its appendices, confirms compliance with these Codes of Practice by British Coal Opencast, and this report was completed for the Plenmeller site in October 1990.

Coal-prospecting drilling was carried out mainly between 1973 and 1981, with some further drilling in 1989. Around 2,400 vertical boreholes were sunk, mostly by open-hole air-flush rotary drilling techniques, plus some rotary coring. Slimline (geophysical) logging (long-spacing density, high-resolution density and natural gamma) was applied down a few boreholes during the 1989 phase of drilling. From this work plans were drawn up for superficial deposits thickness and distribution, rock-head contours, geology and structure, plus detailed geological cross-sections. The Final Geological Report and borehole schedules are the other documents that result from the prospecting activities.

For geotechnical investigation purposes, the first walkover survey took place in 1979, and others continued intermittently through to 1990. Examination of earlier editions of the Ordnance Surveys, statutory undertakers' records, air survey photographs, British Geological Survey records, Mines Records Office documents, and detailed ground surveys resulted in detailed composite plans of 'surface and shallow features' and 'old workings'.

In order to provide soil mechanics design parameters for spoil mounds and excavated slopes, around 30 trial pits were dug by a backhoe machine and 100 boreholes sunk by cable-percussion techniques. These exploratory holes included investigations for the diversion route for the C322 highway, and the various water-treatment areas, but a completely separate ground investigation exercise was carried out for the coal handling and loading facilities.

The geotechnical properties of the bedrock were assessed from rock core samples obtained from 13 fully cored boreholes of various diameters from 76mm (412/HWF) to 112mm (SF). Transparent plastic liners were used and samples of jointing, seat earths and other weak zones were obtained. The joint surfaces were tested using the Golder Richards apparatus[3] and the more plastic (clayey) horizons were tested in the conventional 60mm square Soil Mechanics shear box.

Groundwater investigations involved the installation of piezometers of the Casagrande type. Thirteen piezometers were provided in the superficial deposits or at rock-head, and 10 piezometers were placed into bedrock. All these monitoring installations recorded fairly high water levels.

GEOLOGICAL SUCCESSION AND STRUCTURE

The greater part of the site is covered by a layer of peat which varies in thickness from less than 0.5m up to about 8m. Glacial deposits blanket virtually the whole of the site area and underly the peat. A buried glacial channel starts in the central area and extends eastwards to the eastern boundary of the site where the glacial materials are over 40m thick, and very steep rock-head gradients occur as a result. Throughout much of the western and southern parts of the site the thickness of superficial materials is generally between 1m and 5m, but in the northern part thicknesses of 10m to 15m exist.

In general, the bedrock strata at Plenmeller are the Lower Coal Measures, comprising sandstones, siltstones, mudstones, coal seams and occasional seat earths. The succession to be worked ranges from the Unnamed (KW00) seam down to the base of the Low Main (BW00) seam and includes 12 contractual seams. The occurrence of the Plenmeller coalfield is in the form of an isolated faulted outlier. The Coal Measures within the site are bounded to the south by the Stublick Fault (which down-throws northwards by approximately 250m) and associated splinter faults, beyond which are the strata of the Millstone Grit series. The base of the Coal Measures outcrops in the western, northern and eastern parts of the site, beyond which the Millstone Grits emerge again. The structure is generally steeply dipping with gradients rarely less than 1 in 5, and divides into two separate

excavation areas, referred to as Areas A and B.

Area A (west) is broadly half-basin shaped, very complexly faulted and truncated by the Stublick Fault. A dominant NNW–SSE fault separates this area from Area A (east) where the strata dip steeply, but very uniformly, to the south with a constant east–west strike, until it meets the Stublick Fault.

To the west and straddling the C322 road, the bedrock strata in Area B are folded into an east–west trending syncline closed at both ends by the inwardly pitching axis. Both ends are characterized by fan-shaped dip faulting, and the southern limb is restricted in its width by truncation against the Stublick Fault.

PREVIOUS MINING AND GROUNDWATER

Piezometers were installed on either side of the Stublick Fault and recorded similar water levels, thus indicating that the fault does not act as an aquiclude to the groundwater stored in the strata to the south.

Inspection of the Mining Records Office plans and documents, together with extensive prospecting drilling, revealed that previous underground working had taken place between 1860 and 1942 in the following six seams:

Upper Craig Nook	(HW00)
Lower Craig Nook	(GW00)
Threequarter	(FW00)
Wellsyke	(EW00)
Slag	(CW00)
Low Main	(BW00)

Records showed that a relatively small strip of Slag (CW00) seam had been taken by opencasting at outcrop in the Blackshield Bog area (immediately north of the C322 highway). A total of 94 shafts and 14 drifts were also recorded.

These old underground workings act as a large underground drainage system which collects water from the surrounding water-bearing strata, provides some balancing storage and then feeds it into the Level Sike and Kingswood Burn. The aquifers which feed into the site are generally located to the south and are associated with the Stublick Fault and the up-thrown older strata beyond. The approximate volumes of water stored in Areas A and B are 160,000m^3 and 130,000m^3 respectively. These old workings will be removed as the opencasting operations progress, but the aquifers will continue to feed water into the excavations and backfilled areas.

SITE OPERATIONS

Work on Plenmeller Common began during the spring of 1991.

There are six water-treatment areas around the perimeter of the site, with between one and three lagoons at each WTA, resulting in 15 lagoons in total. The majority have been formed by excavating below surface level into the glacial deposits, but five lagoons have been formed by impounding embankments constructed from glacial clays and damming three of the watercourses draining from the site. These water-treatment arrangements were designed by civil engineering consultants, and the ground investigations involved a total of 88 trial pits in addition to boreholes and trial pits carried out in connection with the Geotechnical Codes of Practice described earlier.

The existing C322 Haltwhistle to Bearsbridge road originally crossed the western part of the site (coaling Area B) in a north-west to south-east direction. This has been permanently diverted over a length of 2km on to a new alignment some 300m further to the north-east, so that coal can be mined from beneath the original route. Most of the new road has been constructed on embankments which first required the excavation of peat and underlying organic clay to a maximum depth of 4m. The whole of the fill was Coal Measures bedrock taken from the initial excavations in coaling areas A and B.

The first bulk excavation and earthmoving operations were the removal and storage of seeds-rich topsoil, peat and subsoil. These storage mounds were mostly sited along the northern sides of the two coaling areas, with two subsoil mounds also sited

Top end of the log row laid for a conveyor route

on the south-west side of Area B. The two main overburden storage mounds are both sited on the south side of their respective excavation areas, and are each limited to a maximum vertical height of 30m by the planning conditions.

Excavation in the initial cut in both Area A and Area B began simultaneously. Overburden removal is performed by two Demag 185S hydraulic excavators, normally both working as backhoes, but the machine in Area B was initially deployed as a faceshovel. These machines are equipped with 13m^3 buckets and are used to load into a fleet of seven Cat 785 dumptrucks of 150 tonnes capacity. Most of the rock overburden is blasted prior to excavation. The average overburden-to-coal ratio for the site is around 20 to 1, whilst the maximum depth of working in Area A is 94m, and in Area B is 51m. Dewatering is effected by pumping from sumps located at the lowest point of the current cut, and the water is pumped to the water-treatment areas (lagoons) prior to discharging into the exiting watercourses.

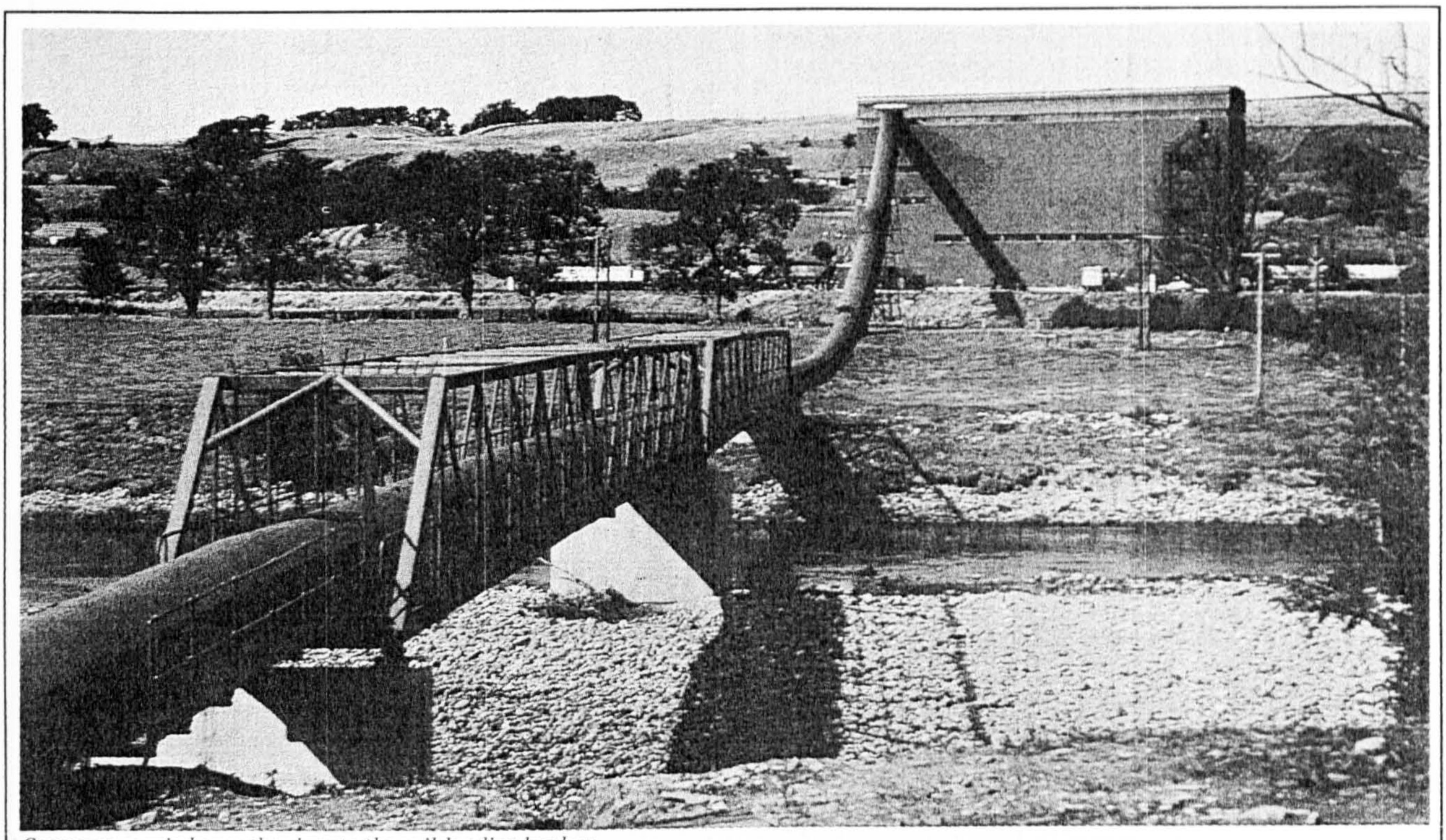
The coal is lifted by a team of up to five smaller hydraulic shovels and loaded into a fleet of articulated dumptrucks which deliver the coal to the screening and crushing plant located in the north-western part of the site. Dozers and graders are used to construct and maintain the internal haul-road system.

RESTORATION

Once coaling and backfilling have been completed, it is intended that the site will be restored to a profile agreed with the mineral planning authority, and to something like its former mode of vegetation. The Environmental Advisory Unit (EUA) of Liverpool University are supervising trials on various plots within the site. The aim is to try as many different options of lime, fertilizer and seed sources as possible, and so determine which combination works best for the specific site conditions and terrain profile.

Work on the moorland restoration began with a survey of the vegetation and the setting-up of the first trial plot in 1990. EUA and ADAS recommended ways of spoil stripping and storage that would assist in the long-term recreation of the moorland habitat.

Topsoil is stored in low-level mounds to create a large surface area of live soil, while the deep-seated peat, totalling 2.3 million



Conveyor carried over the river to the rail-loading bunker

m^3, is stored separately, above and below ground level. After a year, heather was growing on the first of the plots.

A small trial plot has also been set up to recreate a sphagnum bog — the wettest habitat on the site. This pioneering work covering 1ha is carried out by transferring sphagnum moss from the donor location to the newly created area.

COAL PROCESSING, TRANSPORTATION AND LOADING FACILITIES

A principal requirement of the planning permission for the Plenmeller project is that no coal should leave the site by road. As a result, all the coal from Plenmeller is transferred off site by conveyor to the Newcastle to Carlisle main-line railway at Melkridge. The overall cost of these new construction works, as described in this section, was around £5 million.

Coal from the excavation areas is brought by off-road lorries to the crushing and screening plant on the north-western boundary of the site. The structures at this location include a weighbridge, 70-tonne reception hopper, 250-tonne/h screening and crushing plant, workshop and amenity buildings. From here the coal is loaded on to an overland conveyor system of some 3km total length and is transported across moorland, woodland and grazing land. The conveyor consists of four sections ranging from 200m to 2,000m in length with a steel-framed transfer house at each intersection. A minor public highway, a historic garden wall and the river South Tyne exist in close proximity to one another and have to be crossed before the coal arrives at the railhead. This last section involves a 9m deep vertical shaft with a rubber-lined cascade chute, a 42m long by 2.59m internal diameter tunnel, followed by an 84m long twin-span bridge. The final section of conveyor rises steeply to some 20m above ground where it connects with a 2,000-tonne capacity rapid-loading bunker. A new rail siding has also been constructed to facilitate the automated loading of trains.

An examination of the British Geological Survey maps indicated that the bedrocks underlying the conveyor route are the Millstone Grits and the Upper Limestone group of the Upper Carboniferous series. Across the moorland section the superficial deposits are shown as peat and glacial clays, whereas recent river-terrace deposits and alluvium are shown across the river valley section. Around 45 cable percussion boreholes and 10 rotary boreholes were sunk for ground-investigation purposes for these construction works. In addition, peat-probing, hand-augering and trial pitting were also carried out.

Screening and crushing plant

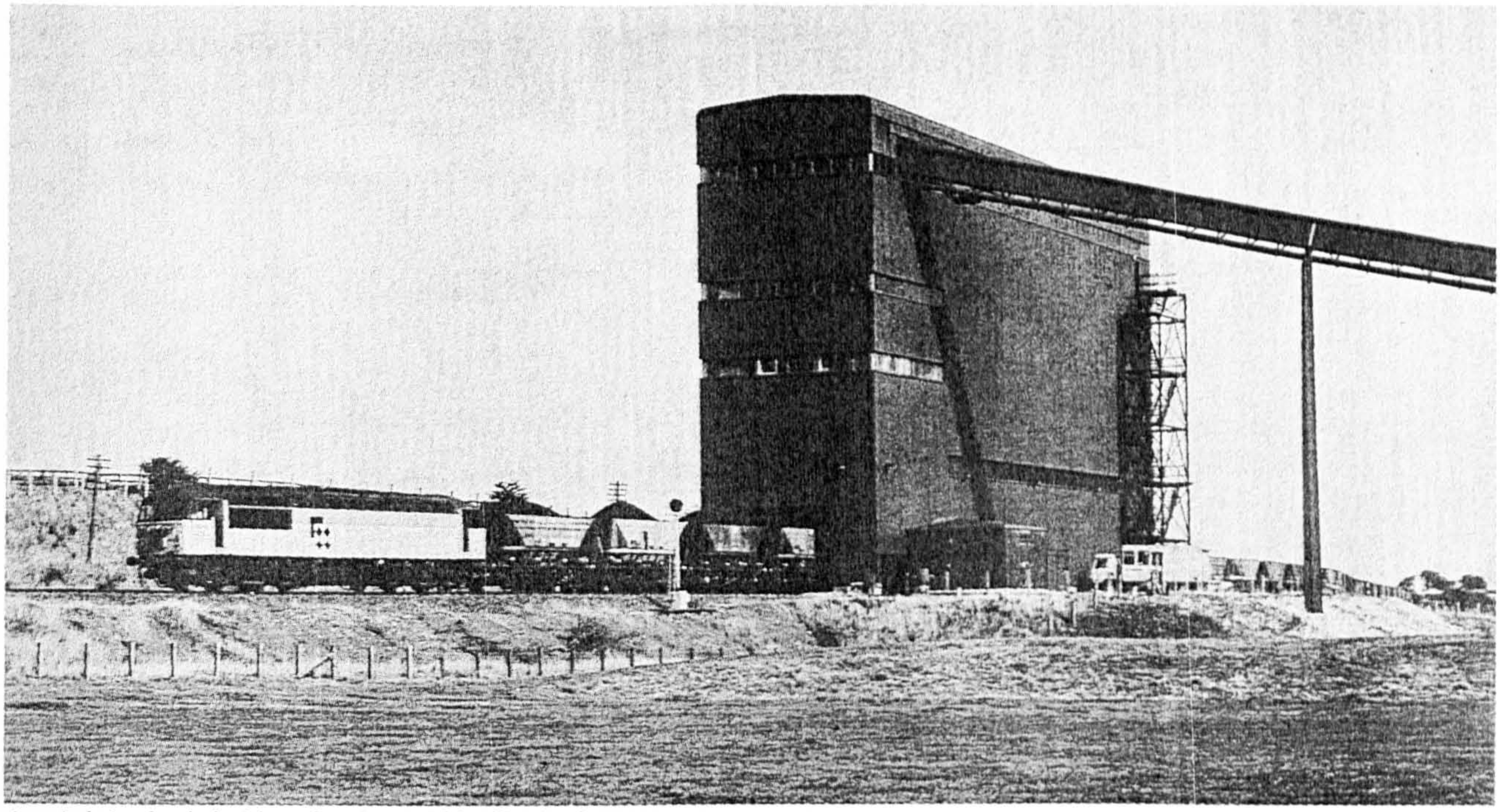
In the area of the crushing and screening plant, the boreholes proved peat thicknesses up to 4m. This peat was all removed from the locations of the structures and spread foundations constructed on the underlying firm-to-stiff glacial clays. The reception hopper was constructed some 7m below original ground level, and this involved excavating through the glacial clays and up to 2m into the underlying mudstones. All excavations were battered rater than close-supported.

Moorland conveyor

The ground conditions underlying the main moorland section of the conveyor include 0.2–0.7m of peat or peaty topsoil overlaying soft mottled clay and stiff glacial clay with bedrock (mudstone, siltstone or limestone) at depths varying from 3.0m to at least 12.0m. The major part of the conveyor route is at ground level, but there are some elevated sections supported on piers. The method used to anchor the structure, particularly through the peaty areas, is by wooden pegs driven to depths up to 2m. This is to ensure that the foundations are easily removed when the site is restored. Similarly, the access road provided alongside the conveyor, to facilitate maintenance and repair, is constructed of removable softwood logs and wire mesh so as to cause minimum long-term damage.

Tunnel and bridge

The final section of the conveyor is from the vertical shaft south of the unclassified Unthank Road to the rail-loading bunker. The ground investigations for the tunnel and river bridge took place during April 1986 when the south bank of the river South Tyne supported a dense plantation of commercial woodland and access into the areas was severely restricted. Hence, a temporary bridge was built across the river which allowed boreholes to be sunk on the proposed foundation locations for the central pier and south abutment. A single borehole was sunk in the



A train being loaded with coal at the rapid-loading bunker

woodland area with a lightweight portable air-winch percussion rig, which proved rock-head at 0.8m below surface level and penetrated weathered sandstone bedrock to 2.1m. A rotary cored borehole was sunk on the location of the vertical shaft at the start of the tunnel, and proved sandstone from 2.3m below surface level down the full depth of the proposed shaft base at 9.0m.

The tunnel was driven northwards from the shaft to the outlet portal on the south bank of the river and passes under the 'historic garden wall' with a clearance of only 3m or so between soffit and foundations. Excavation in the sandstone was mainly by pneumatic tools, but some drilling and blasting was permitted when the tunnel drive was not in the near vicinity of the wall foundations. This wall is now almost derelict, but is of historical interest since it has an in-built central heating system to enable exotic fruits to be grown despite the somewhat harsh Northumbrian climate.

The south abutment of the bridge was founded on sandstone bedrock, rock-head level being at 92.0m AOD under the river bed as compared with 103.3m AOD at the tunnel portal, a fall of 11.3m in about 35m horizontal. The central pier foundations, being in the middle of the watercourse, involved excavation within permanent steel sheet piling driven some 3m into the river-bed gravels and a further 3.5m into the weathered sandstone to be found at 88.5m AOD, and then infilling with mass concrete. Under the north bank, the rock-head level was down at 90.56m AOD and the abutment foundation was constructed on the underlying dense river-terrace gravels at 94.5m AOD. The remainder of the elevated conveyor leading to the rail-loading facilities was supported on piers founded on shallow spread footings.

Railhead

The ground conditions underlying the 2,000-tonne capacity rapid-loading bunker and the new railway siding were proved by both cable percussion and rotary drilling techniques to be river-terrace gravels overlying bedrock of the Upper Limestone group. The exposed excavation faces at a former sand and gravel quarry, located further west along the south bank of the river, showed the river-terrace deposits generally to consist of clayey silty sand and gravel, with pockets and bands of laminated clay of 200mm to 600mm thickness. Thicker deposits of clay occurred in the deeper parts of the excavation (up to 6m deep), and some steeply inclined bands of clay were observed. Also, boulders up to 1.5m diameter were exposed. The cable percussion boreholes at the site of the bunker confirmed the similar nature of the ground conditions, and the SPT values quantified the terrace deposits as dense to very dense. Rotary boreholes proved rock-head at 74.0m to 78.0 AOD (ie some 26m to 30m below ground surface) and the bedrock was recorded as sandstone, siltstones and mudstones with occasional limestones. Two coal seams, the Upper Little Limestone (0.15m thickness) and the Lower Little Limestone (0.73m thickness) were recorded within the range 45m to 55m of ground surface, but no cavities or old workings were encountered. The bunker has been built on shallow spread foundations.

Similar ground conditions were proved by the cable percussion boreholes to underly the whole length of the new rail siding which is located on the south side of the twin-track main line. The construction of the siding necessitated the widening by 5m of the existing 5m high embankment over a distance of 1,100m. A requirement of British Rail was that quarry material had to be brought in by rail from Shap to create an inert firebreak between the existing embankment and the fill for the new siding. Minestone material (colliery spoil heap material) from British Coal's Seaham Colliery was then brought in by rail and compacted in layers to form the new embankment. A farm occupation road running along the southern toe of the existing railway embankment was also reconstructed further to the south.

ACKNOWLEDGEMENTS

The authors wish to thank the directors of British Coal Opencast and Soil Mechanics Ltd for permission to publish this paper. The views expressed here are, however, entirely those of the authors.

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PAPER C17 NORTHUMBERLAND TILL

Robertson, T.L., Clarke, B.G. and Hughes, D.B. 1994.

Classification and strength of Northumberland Till.

Ground Engineering, December 1994, 29-34.

Papers

Classification and strength of Northumberland Till

by TL Robertson, Wardell Armstrong; BG Clarke and DB Hughes, University of Newcastle upon Tyne.

Introduction

Opencast coal mining has taken place in the North East of England for nearly fifty years. Geotechnical investigations for this mining activity were started in the late sixties in order to determine the properties of the overlying superficial deposits. The data from the investigations could be used in the design of soil storage mounds, excavations in overburden and haulage roads. The amount of site investigation has increased over the years especially following the publication of Geotechnical Codes of Practice in 1982 and 1989. These investigations are in addition to the extensive investigations of bedrock conditions.

The site investigation data form a substantial record of the properties of the superficial deposits over a large region of the North East of England stretching between the Rivers Coquet and Wansbeck. Generally the superficial deposits consist of glacial materials. These data are being assessed to allow a model to be developed of the geological and geotechnical processes that have taken place. In this paper the results of a simple statistical study of the classification and strength data are presented.

Sources of information

The area considered, shown in **Figure 1**, covers about 150km^2. Detailed investigations of about 20 sites have been carried out

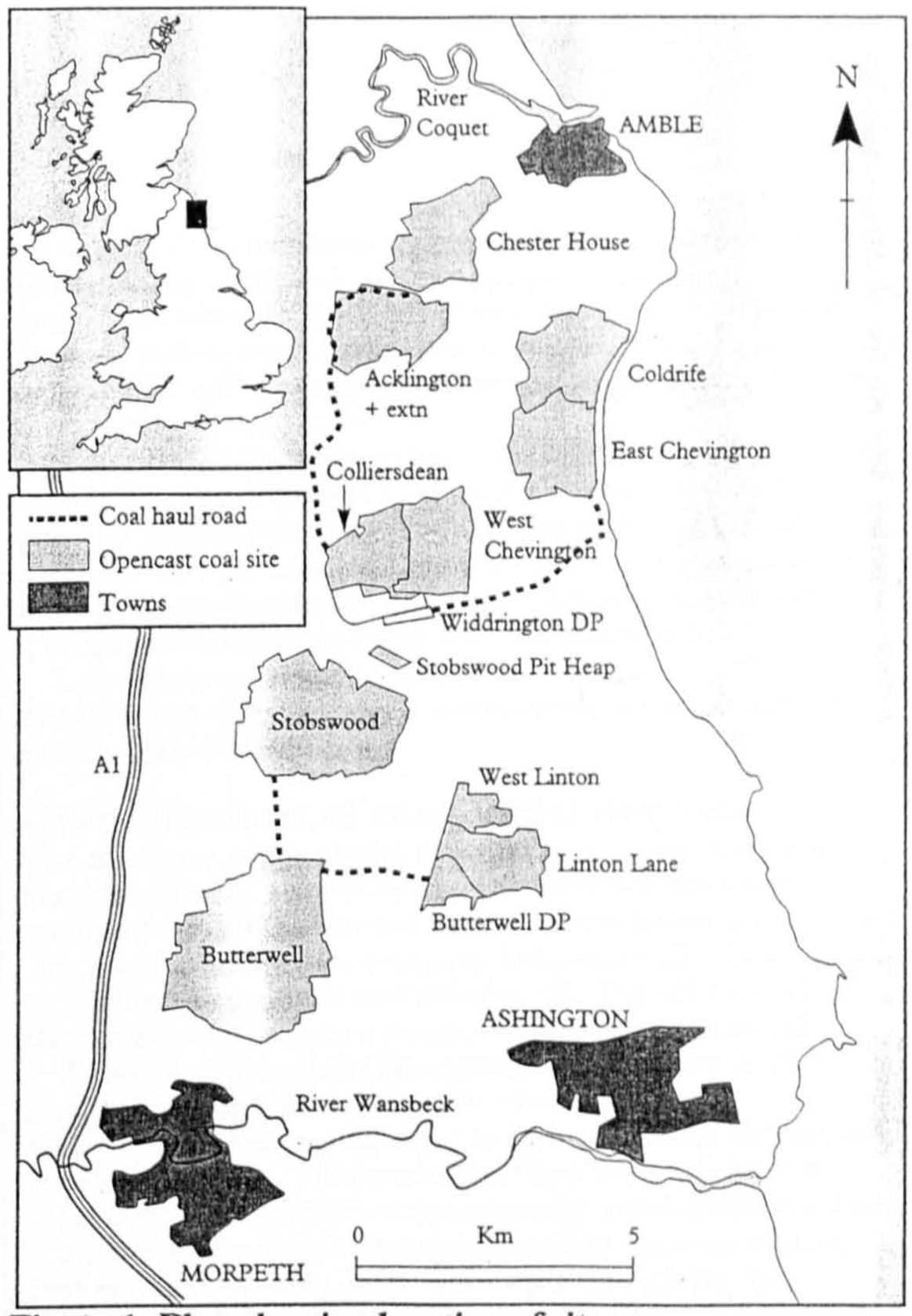

Figure 1: Plan showing location of sites.

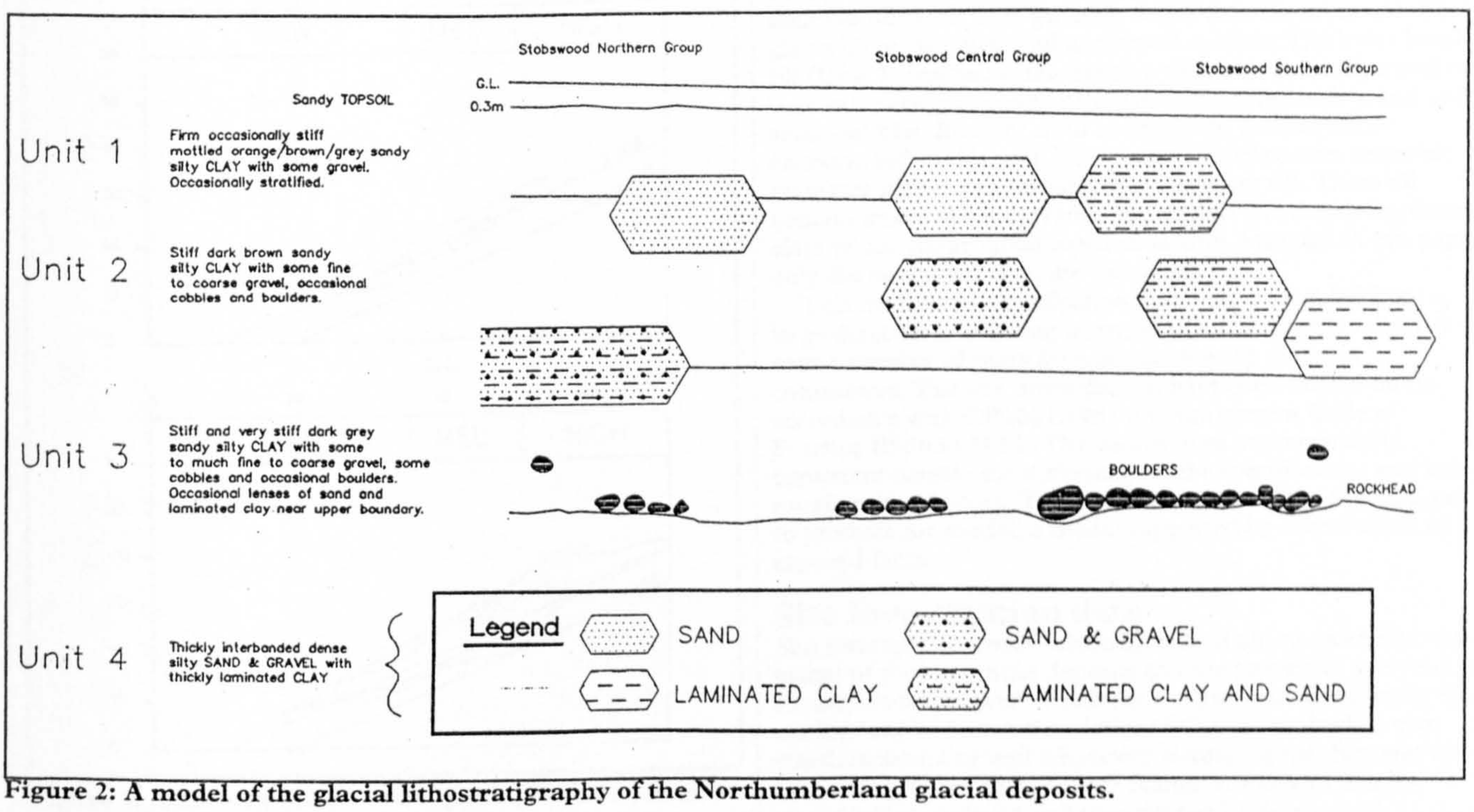

Figure 2: A model of the glacial lithostratigraphy of the Northumberland glacial deposits.

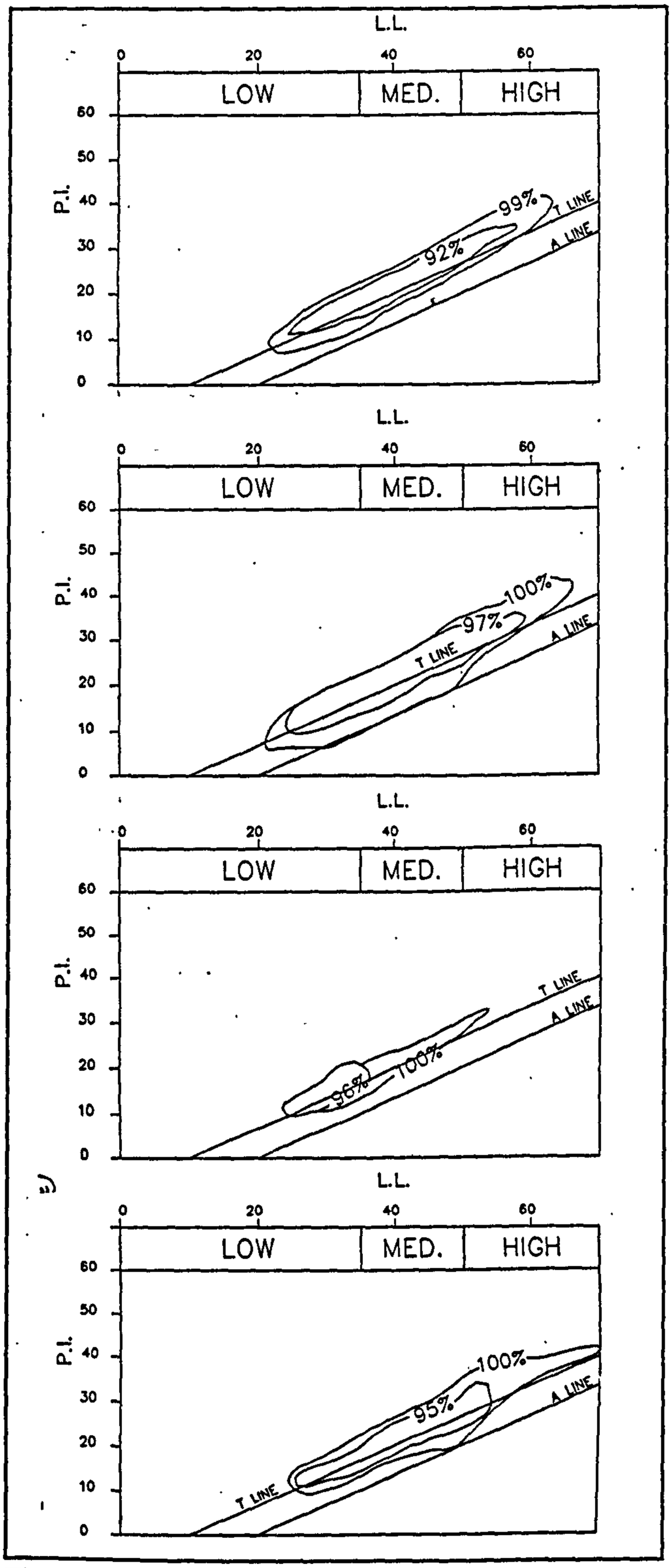

Figure 3: Atterberg Limits of the glacial deposits, from top: Unit 1, Unit 2, Unit 3, Unit 4.

on behalf of the northern region of British Coal Opencast (BCO) using cable tool percussion rigs. These opencast coal production sites are the focus of an infrastructure with coal disposal points, semi permanent coal haulage roads, pit heap reclamation and amenity restoration schemes. Figure 1 shows the location of such sites.

Over 550 boreholes have been sunk and about 50 trial pits excavated. The depths of investigations ranged between two and fifty metres reflecting the thickness of the superficial deposits. The ground conditions are predominantly glacial but do include made ground, peat, dune deposits and alluvium. This paper is only concerned with the glacial deposits.

Laboratory testing was carried out to determine the classification of the soils and assess strength parameters for stability calculations.

Classification of the glacial deposits

The thickness of the glacial deposits varies from 50m to less than 5m. Buried glacial channels do exist but generally rockhead is between ten and twenty metres below ground level. The glacial deposits are primarily lodgement till which can be subdivided into a basal and ablation till. The ablation till is further subdivided into a weathered and unweathered till.

A model of the glacial lithostratigraphy, **Figure 2**, of the Northumberland glacial deposits was built up from observations of excavated faces in opencast mines and the site investigation data. This model has since been validated in successive exposed faces at the currently active Stobswood site.

Four discrete units have been identified ignoring topsoil, peat and alluvial soils. Unit 1, the uppermost layer, is a weathered ablation till which is generally a firm to stiff mottled sandy silty clay with some gravel. The relatively unweathered ablation till (Unit 2) is generally a stiff dark brown sandy silty clay with some gravel and occasional cobbles. The lower basal till (Unit 3) lies below the ablation till and is generally a stiff to very stiff dark grey sandy silty clay containing much gravel and some cobbles. Boulder sized obstructions are frequently encountered in this unit. The percentage of coarser materials increases with depth through the lodgement till. These till contain lenses of sands, sands and gravels and thickly laminated clays which are grouped together as Unit 4 though in this paper only the laminated clays are discussed.

This model and classification data were assembled from a large database containing in excess of 6000 records collected over a number of years from several sites by different contractors. The site investigations have been carried out in accordance with CP2001:1957 and the current Code of Practice BS5930:1981. The descriptions are remarkably consistent despite the differences in the specifications and the number of operators. This justified the use of the descriptions to produce the model, a model supported by observations of exposed faces.

Site investigation data

Site investigations were primarily carried out to determine the extent of the superficial deposits and the properties pertinent to excavation techniques or stability analyses and occasionally for carriageway or foundation design. BCO has evolved its own standard sampling and laboratory testing regime. Samples are taken every metre and at every change of strata so that the classification, stiffness and strength can be determined.

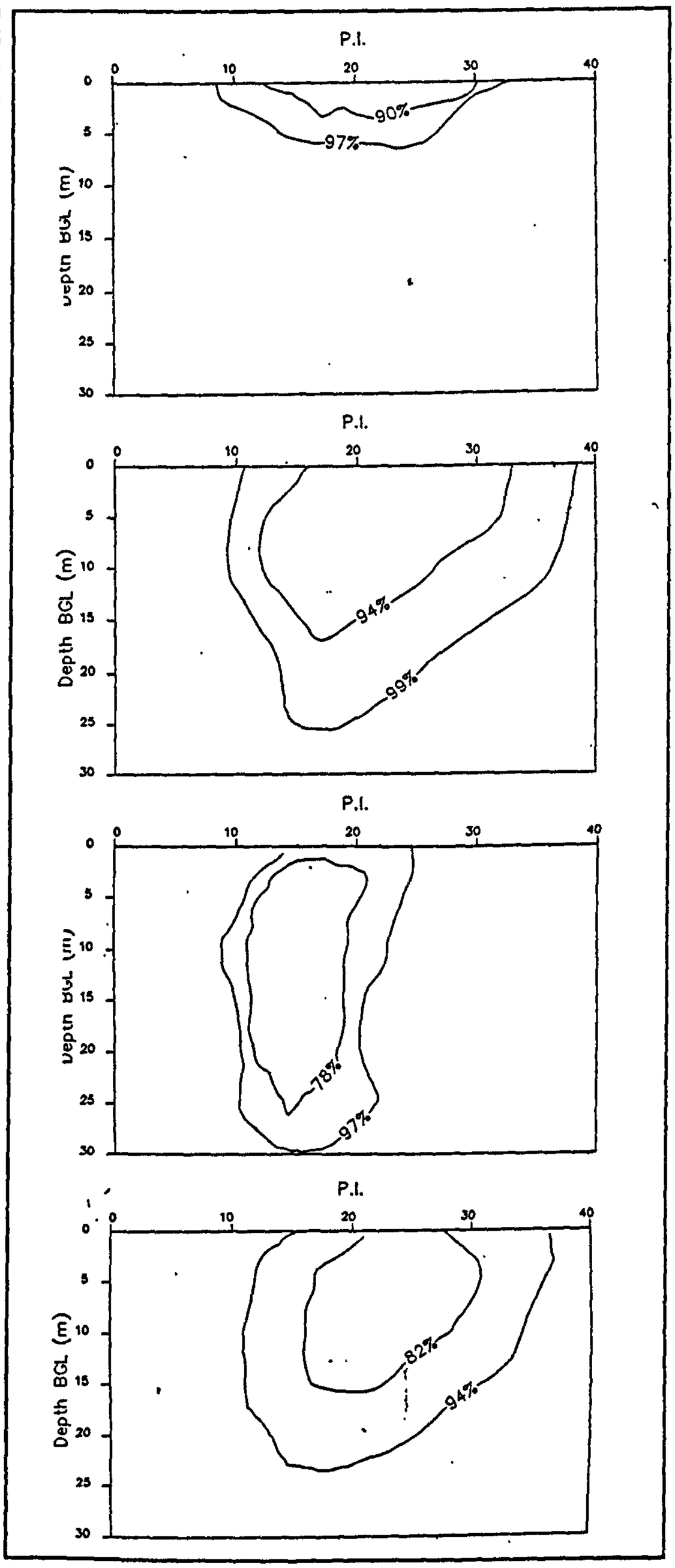

Figure 4: Variation in water content with depth for each unit, from top: Unit 1; Unit 2; Unit 3; Unit 4.

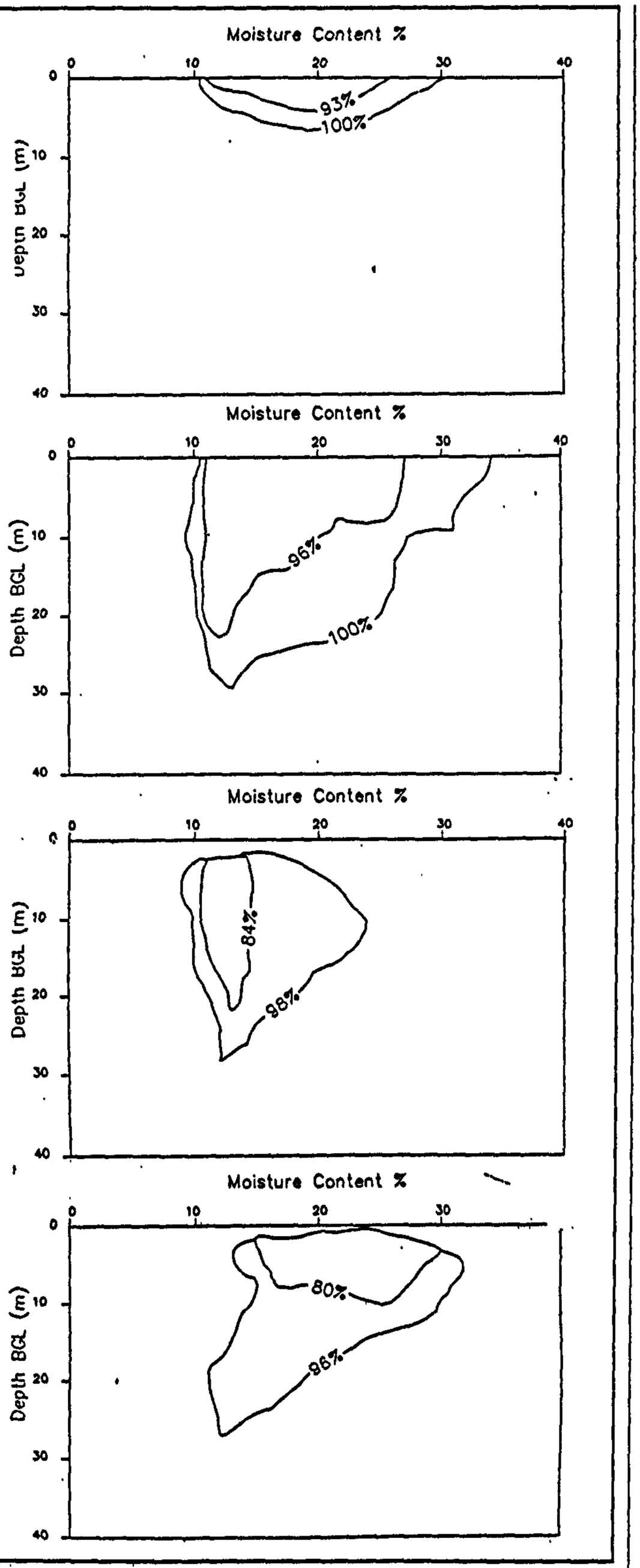

Figure 5: Variation in plasticity index with depth for each unit, from top: Unit 1; Unit 2; Unit 3; Unit 4.

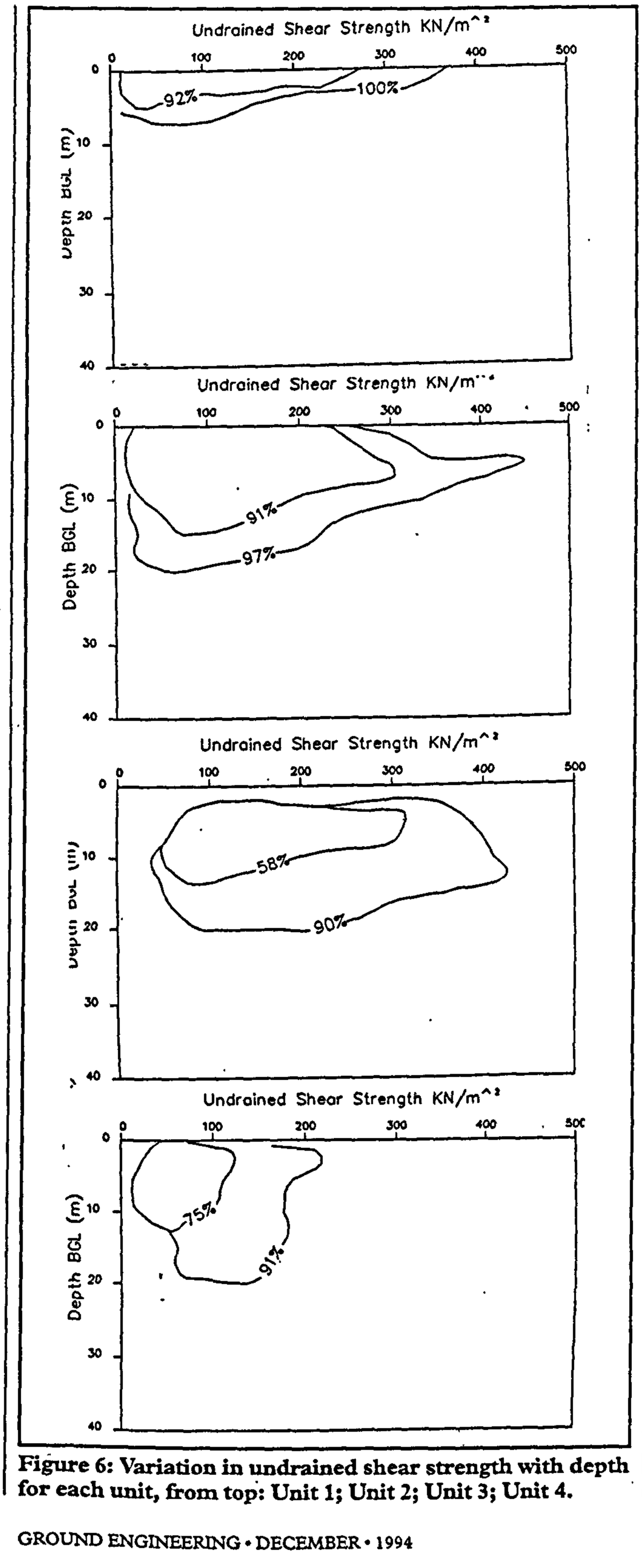

Figure 6: Variation in undrained shear strength with depth for each unit, from top: Unit 1; Unit 2; Unit 3; Unit 4.

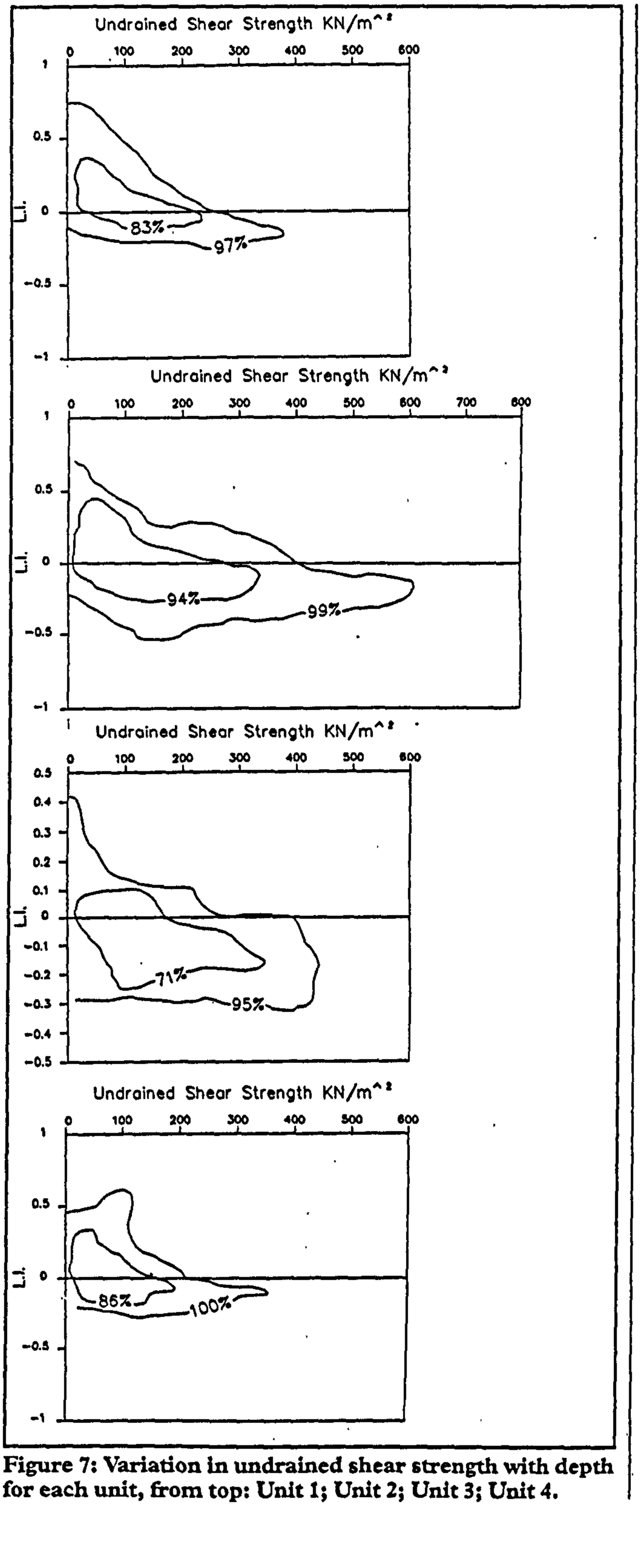

Figure 7: Variation in undrained shear strength with depth for each unit, from top: Unit 1; Unit 2; Unit 3; Unit 4.

The majority of the tests were classification tests, that is tests for water content, Atterberg Limits and undrained shear strength. Undrained unconsolidated triaxial tests are routinely carried out since the results, when used in total stress calculations for spoil mounds, give conservative factors of safety against instability. Many consolidated undrained triaxial tests with pore pressure measurements have been carried out to determine effective strength parameters for stability of excavated faces.

Most tests were carried out on samples from Unit 2 since this is the most common foundation material for soil storage mounds. The strength of Unit 3 is generally, within one profile, greater than that of Unit 2 therefore any postulated failure mechanism will lie above Unit 3. The presence of laminated clay, which tends to be in lenses, can govern the stability of mound or face though it is often difficult to determine whether any laminated clay in a borehole forms a lens which extends over a sufficiently large area to cause concern.

Parameter selection – classification tests

Summaries of the classification data are shown in **Figures 3-6.** The data were assembled on a spreadsheet and divided into the units according to their accompanying descriptions. An indication of the density of data is shown on the Figures using contours. The percentage figure on a contour represents the amount of data within that area. The inner area represents the most densely populated area. As expected there is considerable scatter in the data which reflects the variation in fabric though certain statements and trends can be noted.

Sladen & Wrigley (1983) suggest that data from lodgement tills should lie about the T line on the Cassagrande plot. The data shown here do lie about the T line but the axis of the most densely populated area lies above that line. The tills are clays of medium to low compressibility. In general the basal till is less compressible than the upper ablation till. There is very little difference between the unweathered and weathered ablation till.

The water content is similar to the plastic limit in all cases and generally reduces with depth. The plasticity index also reduces with depth. Note that this refers to the finer fraction of the till. It does not necessarily represent the properties of the till though matrix dominant tills in which the coarse fraction is less than 40% will behave as clays, the effect of the coarser material being negligible.

Undrained shear strength is a function of water content and soil fabric. Several relationships have been suggested in the literature. These include, for overconsolidated clays, a relationship based on overconsolidation ratio and overburden pressure (Ladd et al, 1977); for normally consolidated clays, a relationship based on plasticity index and overburden pressure (Skempton, 1957); and for remoulded clays, a relationship based on liquidity index (Wroth & Wood, 1979).

Details of the groundwater regimes at these sites are not known in sufficient detail, therefore no relationships with effective overburden pressure could be established. Tills generally are insensitive therefore it is reasonable to assume that the remoulded strength is similar to the undisturbed strength. **Figure 7** shows the variation of undrained shear strength with liquidity index. Average lines drawn through the data for each unit are approximately parallel to the proposed relationship for remoulded soils that is

$$c_u \approx 170\, e^{-4.6LI} kN/m^2 \quad (1)$$

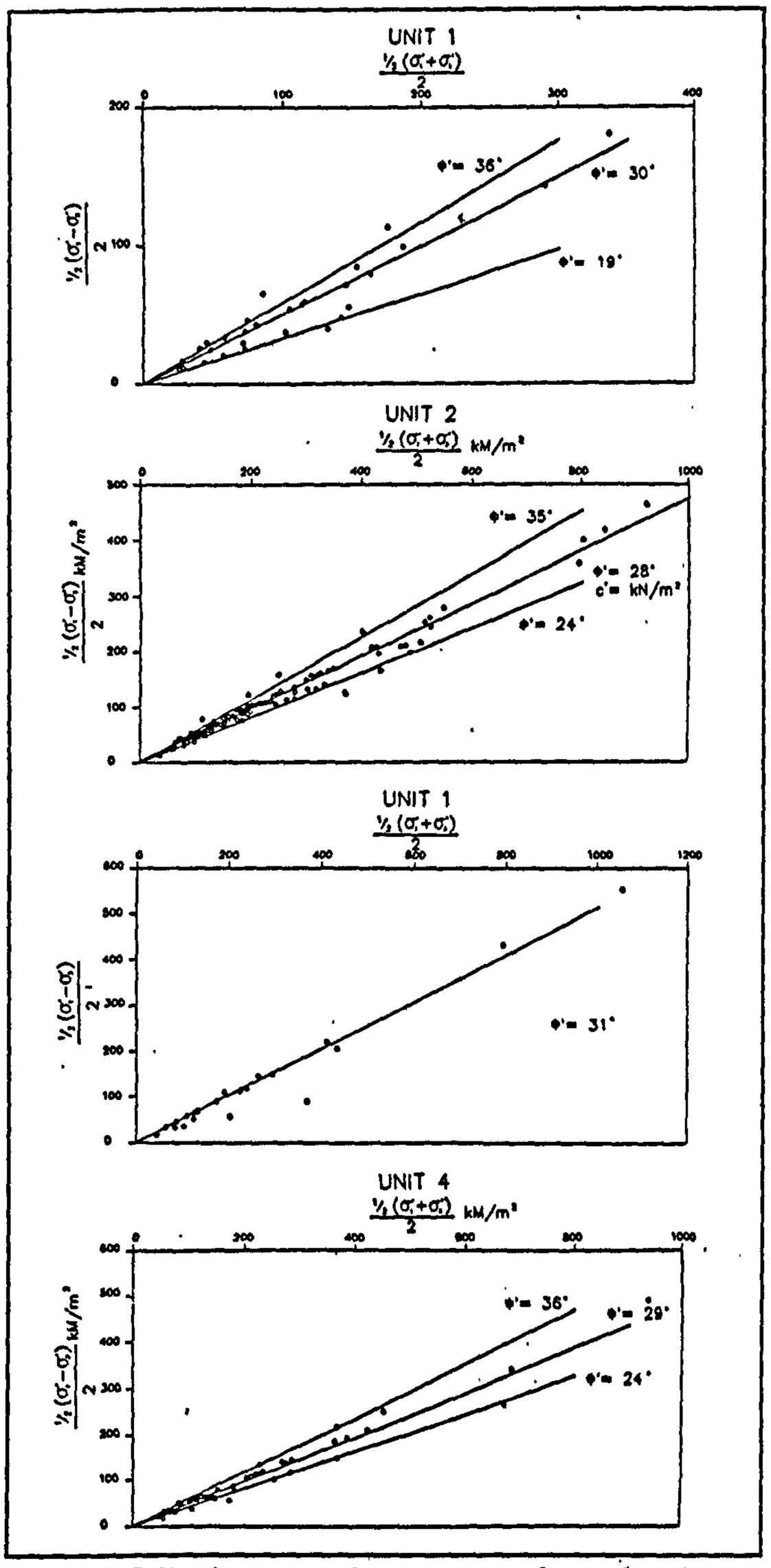

Figure 8: Effective strength parameters for each unit based on the stress state at failure.

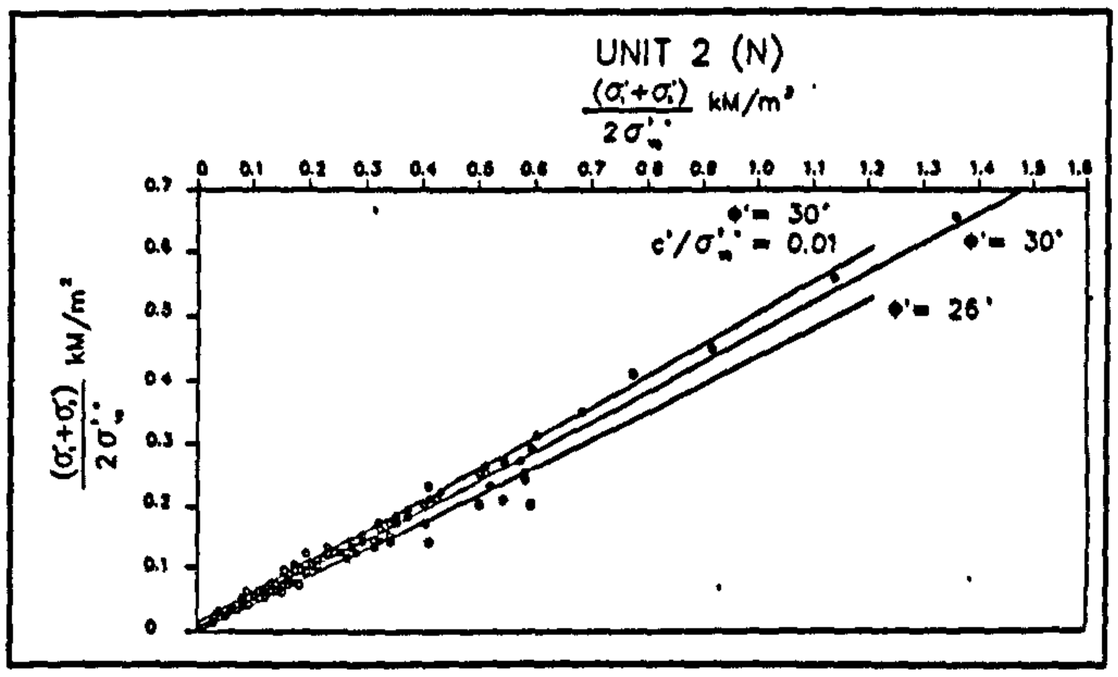

Figure 9: The effective strength parameters for Unit 2 normalised with respect to its intrinsic properties.

Unit	Description	Depth (m)	Water content (%)	Plasticity index (%)	Liquidity index	Dry density (Mg/m³)	Shear strength (kN/m²)
1	mottled clay	0-5	17 (11 - 30)	23 (8 - 36)	0.3 (–0.45 - 0.8)	1.75 (1.50 - 1.96)	150 (30 - 375)
2	red brown till	1-20	14 (9 - 34)	20 (9 - 39)	0 (–0.65 - 0.65)	1.83 (1.62 - 1.93)	180 (50 - 410)
3	grey till	2-50	12 (9 - 23)	17 (8 - 27)	–0.2 (–0.73 - 0.31)	1.97 (1.76 - 2.00)	200 (65 - 410)
4	laminated clay	0-25	20 (10 - 33)	21 (11 - 38)	0.3 (–0.42 - 0.81)	1.74 (1.56-1.92)	100 (50 - 360)

Table 1: Typical values of classification parameters in terms of lithological units (values in brackets show minimum and maximum values).

Depth (m)	Units	Water content (%)	Plasticity index (%)	Dry density Mg/m³	Shear strength kN/m²
0-5	1-4	21	20	1.70	100
5-10	2-4	19	18	1.80	120
0-15	2-4	16	17	1.81	160
15-20	2-4	15	16	1.84	185
20-25	3-4	11	15	1.90	200
25-30	3	10	13	1.93	200

Table 2: Typical values of classification parameters in terms of depth divisions.

However, the liquidity index for a given strength is different between the units and in general reduces with depth (or unit). Sladen & Wrigley (1983) suggest that there is a difference between the water content of the whole sample and that of the sample used to determine the Atterberg limits. This could account for the difference between Wroth & Wood's prediction and the actual data since, as the fraction of coarser particles increases with depth, the water content of the soil will change even though the Atterberg Limits are the same.

Effective strength parameters

Figure 8 shows failure envelopes for the four units. Each data point represents a state of stress depicted by a Mohr's circle at failure. The results for each unit of till lie within a broad band such that the angle of friction is between 19° and 36°. These results indicate that the tills have no cohesion though for a particular set of tests on specimens from one sample there may be a value of cohesion, perhaps up to 25kN/m². The cohesion may be a consequence of a non linear failure envelope rather than a true cohesion. For example, Thorburn & Reid (1973) show for lodgement tills that an increase in cohesion is related to a reduction in angle of friction.

The deviator stress at failure is a function of the consolidation pressure which is related to water content, and the soil fabric. The angle of friction reduces as the plasticity index increases. Figure 8 only shows the variation of deviator stress with mean effective pressure. The scatter may be attributed partly to soil fabric. Burland (1990) suggested that, for natural soils, there exist intrinsic properties which are a function of the physical properties of the clay only. He identified a unique intrinsic compression line for a number of clays with liquid limits ranging between 35% and 128% which is given by

$$\left(\frac{e - e^*}{C^*_c} 100\right) = 2.45 - 1285 \log \sigma'_v + 0.015 (\log \sigma'_v)^3 \qquad (2)$$

where:

e^*100 is the void ratio for 100kN/m² on the intrinsic compression line

C^*c is the slope of the intrinsic swelling line

Figure 9 shows the results of Unit 2 normalised with respect to the intrinsic value of effective stress for the water content and limits of each specimen. The scatter is significantly reduced suggesting that the variation in strength is due to local variations in the physical properties of the till. The normalised curves are based on average intrinsic properties proposed by Burland. Recent tests on these tills have confirmed that their intrinsic compression properties conform with the proposal of Burland.

Applications of the database

This database was developed following the needs of BCO to have preliminary indications of the properties of the coastal till in the Northumberland. It supplements site investigations and can be used as a framework to assess the quality of data and identify the units present in any borehole.

Table 1 gives summary of all the results of the classification tests showing the average values and range of values. **Table 2** gives summary of the variation in properties with depth. Note that the average shear strength and range of strengths increases with depth.

An estimate of effective strength parameters can be obtained from Figure 8. A better estimate can be obtained from **Figure 9** if the water content and limits of the specimen are known.

Conclusions

Site investigation data and geological records have been interpreted to produce a database for coastal tills in the Northumberland. Four main Units have been identified from which a model has been produced. This model conforms with observed excavations.

Profiles of data for each Unit have been prepared. The trends are similar to those published elsewhere though with a larger database it is possible to note significant differences between the different tills. These can be used for an initial assessment of properties of till in the region and as a framework to assess the quality of data from any additional site investigation. This database does not replace good quality site investigation which should be routinely carried out for any engineering project. It supplements site investigation data.

Acknowledgements

The authors wish to thank the directors of BCO for permission to publish this paper and NERC which funded TL Robertson.

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PAPER C18 NORTHUMBERLAND FILL DISCUSSION

Robertson, T.L., Clarke, B.G. and Hughes, D.B. 1995.

Reply to discussion by R.D. Boyd on the paper – Classification and strength of Northumberland Till.

Ground Engineering, April 1995, 39-41.

discussed, and are believed to extend beyond the 'traditional' conditions observed in cyclic triaxial tests which have known and constant total stresses. Pore pressure accumulation is also believed to occur as a result of total stress changes caused in the soil by cycling ('shakedown'), and as a result of redistribution of force away from the skirt tip and into the base plate.

An analytical procedure for calculating the cyclic undrained bearing capacity was presented based on defining a cyclic undrained strength profile throughout the soil as a function of the initial soil stresses, number of cycles and relative density. This strength profile was then integrated using 3D finite element methods for any combination of cyclic vertical, horizontal and moment loads. For analyses of both the prototype and model tests, three undrained cycles of equal magnitude were used as representative of the actual load history after accounting for partial drainage effects. Typical results of such an analysis, correlating well with model test results, are shown in **Figure** 5. One of the interesting features of these results is that regardless of the static load, it is necessary to cycle into tension before a cyclic failure will occur. These results also show that for a constant cyclic load amplitude, the factor of safety against failure increase with increasing static load, from which it may be concluded that the bucket foundation concept should work better with heavier platforms.

Finally, an alternative installation scheme was presented for the future, based on jetting rather than suction and utilising a double skinned skirt. It was suggested that this would be cheaper and structurally more efficient than the current arrangement using suction assistance.

Before the discussion, Nick Ramsey from Fugro gave a short presentation on the offshore model tests. The test results presented support the mechanisms of behaviour described above.

Due to delays in commencing the meeting, only a brief discussion followed.

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Discussion

Discussion by RD Boyd, Halcrow Scotland, on the paper **Classification of strength of Northumberland Till** by TL Robertson, BG Clarke & DB Hughes, published in *Ground Engineering* December 1994.

The paper is an interesting contribution to the topic of glacial till soils and much of the data presented accords well with my own experience for many Scottish tills. The following observations are submitted.

Classification

Accurate classification of north British tills is notoriously difficult, not aided by lack of clarity as to how to apply the current BS5930 1981 in these soils. It is noted that the descriptions used in Figure 2 are not in accordance with the standard. It would be helpful to the reader If the constituent fractions were shown plotted on ternary diagrams as indicated by Dumbleton[1].

Gross structure

Figure 2 suggests a gross structure within the till. This accords with my own experience particularly in lodgement till deposits where pockets, bands and veins of dissimilar soils may be distributed unevenly within a much more regular lithology when viewed at a large scale. This gross structure is most important since it not only adds to the difficulty of strata description and classification based on small samples but can be the cause of engineering problems in that

a) often groundwater flow is associated with more permeable layers which may or may not be interconnected, and

b) strength related behaviour such as slope instability may be associated with the orientation of weaker bands, as noted also in the paper.

Geomorphology

Glacial geomorphology is a complex topic crossing the disciplines of glaciology, geology and soil mechanics. What is needed is a set of defined terms to aid in mutual understanding. As an example, in the section on classification of the glacial deposits, the tills are described as 'primarily lodgement till which can be subdivided into a basal and ablation till'. I find this terminology slightly confusing and would put forward the following definitions after Ashley et al[2].

Lodgement till: Till deposited from the sliding base of a dynamically active glacier by pressure melting and/or other mechanical processes.

Melt-out till: Till deposited by the slow release of debris by melting of glacier ice that is not sliding or deforming internally. It may occur from a wholly stagnant glacier or from slabs of debris-rich ice that stagnate beneath an active glacier.

Basal: Refers to the zone of the glacier near to the bed and with high debris concentration.

With these definitions my interpretation is that the referenced section of text would read 'primarily basal till which can be subdivided into a lodgement and a melt-out till'. The authors views on this would be helpful.

Moisture content

Moisture content is an interesting parameter in glacial tills but careful definition is needed as to the precise test used and the size of the sample tested. It is unclear from the paper how moisture content was determined and it would be helpful if this could be clarified.

Whole (large) sample moisture content is a combination of the absorption of clasts and the so called matrix moisture content, although there is some debate as to what fraction makes up the matrix. If the clasts have low absorption, as in many tills, the whole sample moisture content is generally less than the matrix moisture content. However, if clasts have high absorption as in some sandstones, the opposite may be true. In my experience, simple screening at 20mm and taking the moisture content on the liner fraction has reasonable precision, whereas attempting liner screening leads to lower precision. Clearly acceptance of a standard procedure for determining moisture content in tills would have benefits.

The 20mm liner fraction is used for MCV testing on Scottish tills and standardising on the moisture content of that fraction would also aid assessment of acceptability as fill.

Undrained shear strength

In my experience undrained shear strength is not a useful parameter for analysis in glacial till soils. Invariably test results show a wide scatter with samples often being visibly disturbed because of clasts being encountered. Sometimes only the liner facies can be recovered tending potentially to bias the interpretation where such layers

may make up only a fraction of the whole deposit. Lodgement tills in particular tend to show high undrained shear strength in the range 50-300 kN/m² and clearly are overconsolidated. Figure 7 accords well with my experience of Scottish tills.

I believe though that there are two main reasons for the scatter, over and above sample disturbance.

Dilation: This can be observed in the consolidated undrained triaxial test. Typically excess pore water pressure rises to a maximum at about 2-3% strain and is usually coincident with the maximum effective stress ratio. Thereafter typically it either remains constant or fails by varying amounts often reaching negative values at 20% strain. In effective stress terms, fairly consistent behaviour is observed up to yield giving consistent sets of drained strengths typically as shown on Figure 9. However, the post yield behaviour is far from consistent and whether this is due to the test method or to some inherent variability in the fabric of the sample and hence its dilatancy is unclear. I tend to favour the variability argument. In the unconsolidated undrained triaxial test the effective stress state is unknown, the sample may be partially saturated and the pore pressure response is not known. In the great majority of cases no definite yield point is indicated. Typically the deviator stress continues to rise with strain (indicating dilation). The undrained shear strength is conventionally taken as half the deviator stress at an arbitory strain (usually 20%) and is thus only an index of strength. Perhaps strength indices at say 5% and at 20% strain, and the ratio between the two might be more useful. I submit that dilation behaviour is very fabric dependent and probably stress history dependent and inherently highly variable in these soils.

Variable consolidation pressure: Most lodgement tills, as they were laid down were squeezed and bulldozed into place by the glacier, under conditions of very high total stress, very high shear stress and high and variable pore pressures.

The maximum consolidation pressure is conventionally taken as the maximum difference between the total stress and the corresponding pore pressure which existed under the glacier, but at extremes of stress, and in highly dynamic conditions this may be complex. Perturbations in stress and variable rates of dissipation of excess pore pressure may have resulted in variable overconsolidation pressures at the sample scale, which manifest themselves as variable undrained shear strength (variable dilatancy). A review of consolidation tests in tills typically will indicate a difficulty in identifying a clear preconsolidation pressure, with a large variation in interpreted values often being experienced on supposedly similar samples.

I submit that it is drained not undrained strength which controls behaviour in the vast majority of situations involving till soils. The great difficulty is in predicting when such situations will be manifest in a deposit which generally shows considerable variation in permeability and patterns of drainage.

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Reply to the discussion by RD Boyd on the paper **Classification and strength of Northumberland Till** by the paper's authors TL Robertson, BG Clarke & DB Hughes

The authors are grateful to RD Boyd for his observations and helpful comments on their paper, Much of Mr Boyd's discussion appears to accord with the authors' views and findings, but there are some points which must be clarified. This is attempted using the same headings as in Mr Boyd's discussion.

Classification

It should be noted that the data used in this exercise were supplied by the former British Coal Opencast Northern Region, and consisted of ground investigation reports for about 20 individual mining/construction projects carried out between 1967 and 1989. Hence some of the work was in accordance with CP2001:1957 and some to BS5930:1981. Unit descriptions used in Figure 2 are both combined and abbreviated.

The individual ground investigation projects were not carried out with any future research application in mind, and therefore insufficient data was gathered on particle size distribution to produce ternary diagrams. Eyles & Sladen (1981) reported results of 213 particle size distribution as envelopes where their zone one corresponds to our unit three and zones three and four to units two and one respectively. Ongoing studies of the properties of Northumberland Tills will produce additional data to compare with their findings.

Geomorphology

Smith (1971) and others consider the lower grey till (unit three) and the upper red till (units one and two), both lodgement tills, were formed by separate periods of glaciation, Eyles & Sladen (1981) suggest that a lodgement till was deposited in a single phase of glaciation; the upper red till being a post glacial weathering profile up to 8m thick. They attribute the presence of sand, gravel and laminated clay as infilling of subglacial channels, cavities and lakes. The significant difference in sand content is

attributed to weathering.

Our study has shown that sand, gravel and laminated clay occur very infrequently in unit three, and can be extensive in units one and two. A possibility is that the upper till is either an ablation moraine or a melt out till.

There are, thus, three theories 'for the deposition of these tills which give rise to either two lodgement tills or a single lodgement till weathered in the upper zones or a single lodgement till overlain by a melt out till. (Current studies of Northumberland Till based on extensive records will clarify which is correct). We consider the last description may be applicable but we agree with Mr Boyd that a consistent set of terms is required,

Moisture content

The ground investigation projects used in this research exercise were carried out over many years and by several different contractors. The usual procedure for testing samples was to extrude 200mm × 100mm diameter cylinders which were weighed before and after triaxial testing, and then after oven drying to BS1377, to obtain before and after test moisture contents. Generally there was no removal of clasts. Atterberg limit material was taken from the same U100 sample and sieved 'through a 425μm sieve (as per BS1377).

The clast materials in these tills are mainly Coal Measures sandstones and mudstones as occur at rockhead throughout the region, but there are also significant amounts of igneous rocks originating from the Cheviots and the Lake District.

In Northumbrian opencast coal mining uncompacted backfilling (end tipping or dragline casting) is the norm, and the assessment of the excavated spoils as "suitable/marginal/unsuitable" as in highway construction projects is not usually required. Hence the determination of matrix moisture content is not as systematic as the determination of bulk moisture content. Similarly, compaction testing and MCV determination is only carried out at isolated locations where say some minor highway works arise as a consequence of the mining project.

Undrained shear strength

We agree with Mr Boyd that dilation behaviour is fabric and stress history dependent. Within any particular deposit where the stress history may be similar throughout the deposit the fabric could be variable. Further, the specimens that are tested, which do not necessarily represent the in situ fabric, are specially selected so that tests can be carried out.

If conventional theories of soil behaviour apply to till then the undrained shear strengths measured in the laboratory would suggest a heavily over consolidated soil. This is often taken to be a consequence of the weight of ice (eg Boulton & Paul, 1976) but more recently it is attributed to high pore pressures if the till is underlain by an aquifer (Boulton & Dobbie, 1993). These pore pressures are created by the melt water at the base of the ice being forced through the till under constant pressure due to the weight of the ice. The stress changes that occur are complex and as yet may not be fully understood.

Mr Boyd considers that drained rather than undrained strength controls the behaviour of tills. In a short term stability problem such as foundation loading or excavations this will be correct if either the mass permeability is high enough or the till is sufficiently stiff enough or the till is partially saturated. As the stiffness of the till increases the rate of dissipation of pore pressure increases provided there is little change in the coefficient of permeability ($c_v=k/(m_v\gamma_w)$). Thus the 'undrained' strength measured may actually be a 'drained' strength. This suggests there may be a gradual change from an undrained to a drained strength as the stiffness of the till increases.

We hope these comments are of assistance.

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